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OF THE
AMERICAN CONCRETE INSTITUTE

ACI
PROCEEDINGS
V. 57

Report of the Committee on Nominations appears on p. 3 of the News Letter

P/S Concrete "Sky-Hooks"

150-ft x 390-ft. gymnasium of the Central Washington College of Education, Ellensburg, Wash. Prestressed concrete pylons were made with 'Incor' 24-hour cement.

This floating roof covers an acre-and-a-third without a supporting column, thanks to the ingenious use of 80-foot-tall prestressed concrete pylons. These delicately-tapered columns suspend the slip-fastened roof via 1-5/16" bridge cables. The design allows for 28-psf dead load, 30-psf snow loading and as much as 100-mph horizontal winds.

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Engineered by Anderson-Birkeland-Anderson and erected by Roy T. Earley Company, the hollow pylons were precision-cast in half sections by Concrete Technology Corporation (all of Tacoma). Ralph Burkhard (Seattle) was the architect. Ready-mixed concrete was furnished by Ellensburg Cement Product Company. Dependable 'Incor' 24-hour cement was used throughout.

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Considerations involved in the preparation of specifications for "Concrete and Concrete Materials for Glen Canyon Dam" are disclosed by author trio WALTER H. PRICE, L. P. WITTE, and L. C. PORTER.

J. G. MACGREGOR, M. A. SOZEN, and C. P. SIESS discuss the "Effect of Draped Reinforcement on Behavior of Prestressed Concrete Beams."

ADRIAN PAUW comments on "The Effect of Aggregate Density on the Static Modulus of Elasticity of Concrete."

MANLEY W. SAHLBERG reports on a testing project revealing that "Reconsolidation Technique for Concrete Proves Effective in Tests on Wall Panels of Masonry Forms Using High Lift Grouting System."

O. P. JAIN presents a proposed method for ultimate load theory calculations in his paper entitled "Ultimate Strength of Reinforced Concrete Arches."

J. TAUB and A. M. NEVILLE conclude the five-part series on "The Resistance to Shear of Reinforced Concrete Beams. Part 5—Anchorage and Bond."

Discussion of papers published in the April, May, and June, 1960 issues will appear in the December JOURNAL in accordance with the Institute's quarterly publication schedule for *Proceedings* discussion.

On the Cover—Stadium of the Sporting Club of Portugal, Lisbon. A brief description of the structure, as designed and of the modifications subsequently suggested by the contractor, appears in Concrete Briefs. — Photo courtesy Laboratorio Nacional de Engenharia Civil, Lisbon, Portugal

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Design Criteria for Reinforced Columns under Axial Load and Biaxial Bending

By BORIS BRESLER

Several design criteria for columns subjected to compression combined with biaxial bending are discussed. The approximate load carrying capacity is defined in terms of easily determined parameters without the cumbersome trial and error procedures.

■ THE CRITERIA GENERALLY proposed[1-5] for determining ultimate strength of reinforced concrete members subjected to compression combined with biaxial bending are based on limiting the maximum strain (or stress) in the concrete to some prescribed value. Ideal nonlinear stress-strain laws for steel and concrete, conservation of plane sections, no slip, and no tension resistance by concrete are usually assumed.

The load carrying capacities discussed here apply to relatively short columns for which the effect of lateral deflections on the magnitudes of bending moments is negligible. Furthermore, effects of sustained load and of reversal of bending moments are not considered.

When the position of the neutral axis is known or assumed, the magnitude of the load P_u and the components of bending moments M_x and M_y which result in the prescribed limit strain, can be determined using equations of equilibrium. When the position of the neutral axis is not known, the equations of equilibrium can be solved only by the method of successive approximations. All such procedures involve more or less tedious cycles of numerical calculations.

The criteria proposed in this paper are based on approximations of "surfaces of failure" which are defined as surfaces obtained by plotting the failure load P_u as a function of its eccentricities x and y or of the components of bending moment M_x and M_y (Fig. 1, 2, and 3).

FAILURE SURFACES

The magnitude of the failure load P_u acting on the column with eccentricities x and y depends principally on the column dimensions, amount and distribution of steel reinforcement, stress-strain characteristics of steel and concrete, and on such secondary factors as amount of concrete cover and arrangement and size of lateral ties or spiral. The mathematical expressions required to define the failure load appear

ACI member **Boris Bresler** is professor of civil engineering, Department of Civil Engineering, University of California, Berkeley. He has been on the university faculty since 1946 and has been involved in many and varied research programs. He is a member of ACI-ASCE Committee 326, Shear and Diagonal Tension.

to be so complex that an analytical formulation is not possible at this time. It is assumed here that such an expression would result in a function which would define a surface S_1 (P_u, x, y) as shown in Fig. 1.

This basic surface can be transformed in various ways. For example, a "reciprocal" surface can be derived from S_1, wherein the reciprocal of the failure load P_u is used, so that the surface S_2 $(1/P_u, x, y)$ appears as shown in Fig. 2. Another surface can be obtained by relating the failure load P_u to moments $M_x = P_u y$ and $M_y = P_u x$, so that a surface $S_3 = (P_u, M_x, M_y)$ appears as shown in Fig. 3. It can be seen that the traces of surface S_3 on the $M_x = 0$, and $M_y = 0$ planes are the familiar *P-M* interaction curves.

While exact mathematical expressions defining the failure surfaces cannot be established, some approximations can be derived. An approximation based on the surface S_3 was recently suggested by Pannell[6] which proposed that an equivalent moment M_y about the major axis y replace the two bending moment components M_x and M_y. The moment M_y is defined as:

$$M_y = K M_y \quad \text{.....(1)}$$

where K is a coefficient depending on M_x/M_y, section shape, amount and distribution of reinforcement, and steel cover ratio. The derivation of this coefficient has not been included in the reference cited and thus its validity cannot be fully evaluated. Calculation of this coefficient K requires three additional functions which are defined by graphs, as apparently they cannot be readily defined by simple mathematical equations. Furthermore, it appears that the values of K are defined

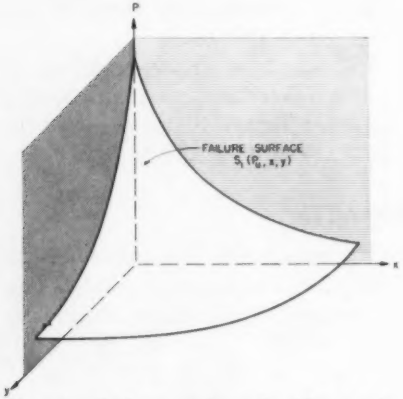

Fig. 1—Failure surface S_1 (P_u, x, y)

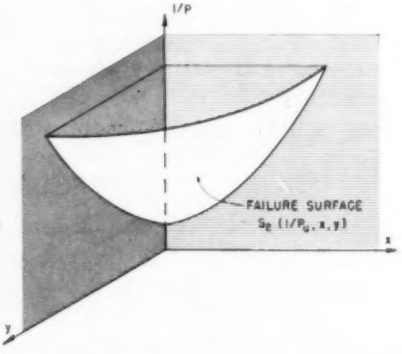

Fig. 2—Failure surface S_2 $(1/P_u, x, y)$

only for cases when nearly equal amounts of steel in each face are used. While the final evaluation of this method must await its full publication, it seems that its use is limited to only those cases for which values of K have been derived and plotted.

Two alternative approximations are discussed here, one of which appears to be remarkably simple and accurate.

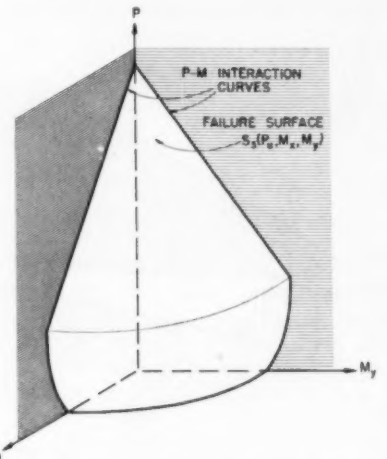

Fig. 3—Failure surface S_3 (P_u, M_x, M_y)

Method A

The ordinate $1/P_u$ on the surface S_2 $(1/P_u, x, y)$ can be approximated by a corresponding ordinate $1/P_i$ on the plane S_2' $(1/P_i, x, y)$, Fig. 4. The plane S_2' is defined as one passing through three characteristic points (Fig. 4):

$$\text{A}\left(\frac{1}{P_x}\ x_A,\ 0\right),\ \text{B}\left(\frac{1}{P_y}\ y_B,\ 0\right),\ \text{and C}\left(\frac{1}{P_o},\ 0,\ 0\right)$$

For a particular column, the value of P_o represents the load carrying capacity under pure axial compression; P_x and P_y represent the load carrying capacities under compression with uniaxial eccentricities x and y, respectively. Thus, for a given column, point C is independent of eccentricities, point B depends on eccentricity y only, and point A depends on eccentricity x only. The loads P_o, P_x, and P_y can be determined by established methods.[3,7,8]

For every point on the surface S_2 $(1/P_u, x, y)$, there is a corresponding plane S_2' $(1/P_i, x, y)$. The approximation of S_2 involves an infinite number of planes, each one applicable only for particular values of eccentricities x and y, and thus each plane defines only one point $1/P_i$ which serves as an approximation to $1/P_u$.

The expression for $1/P_i$ can be easily derived as follows. Let $z = 1/P_i$ corresponding to particular values of x and y; then the plane determined by the three characteristic points A, B, and C (Fig. 4) is defined by the following equation:

$$\left[x - x_A + \frac{x_A}{y_B}\,y\right] + \frac{z_B - z_A}{z_C - z_B}\left[x - x_A + \frac{x_A}{z_B - z_A}\,(z - z_A)\right] = 0 \quad \text{.....(2)}$$

The ordinate z_i on the plane corresponding to $x = x_A$, $y = y_B$ is found from Eq. (2).

$$z_i = z_A + z_B - z_C \quad \text{.....(3)}$$

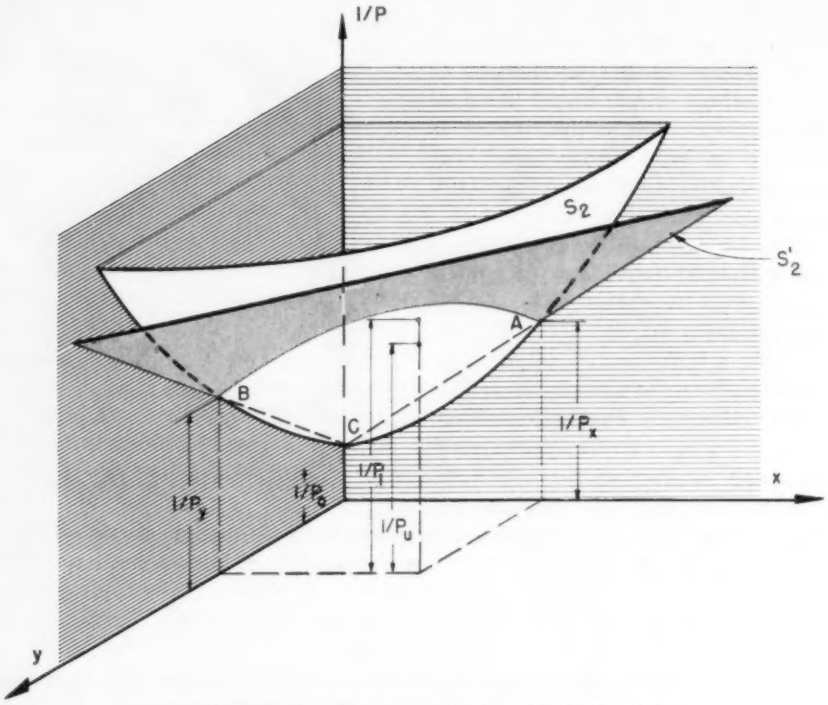

Fig. 4—Graphical representation for Method A

or

$$\frac{1}{P_i} = \frac{1}{P_x} + \frac{1}{P_y} - \frac{1}{P_o} \quad \text{(4)}$$

This approximation has the following advantages:

(1) It is simple in form.

(2) The parameters P_o, P_x, P_y, are determined in a relatively simple manner.

(3) The method appears to be quite general, at least for those common shapes of columns and arrangements of reinforcement for which the point of the surface S_2 can be approximated by a point on the plane S_2' (Fig. 4).

A formula similar to Eq. (4) is given in the Russian specifications,[9] but its derivation could not be found in the Russian textbooks[10,11] or in Russian technical literature available to the author.

Method B

This method is based on approximating surface S_3 (P_u, M_x, M_y) by a family of curves corresponding to constant values of P_u (Fig. 5) which may be thought of as "load contours."

The general form of these curves can be approximated by a non-dimensional interaction equation:

$$\left(\frac{M_x}{M_{xo}}\right)^{\alpha} + \left(\frac{M_y}{M_{yo}}\right)^{\beta} = 1.0 \quad \ldots\ldots (5)$$

where

$$M_x = P_u y,\ M_{xo} = P_u y_o \quad \text{when} \quad x = M_y = 0;\ M_y = P_u x,\ M_{yo} = P_u x_o$$

when $y = M_x = 0$; and α and β are exponents depending on column dimensions, amount and distribution of steel reinforcement, stress-strain characteristics of steel and concrete, amount of concrete cover, and arrangement and size of lateral ties or spiral.

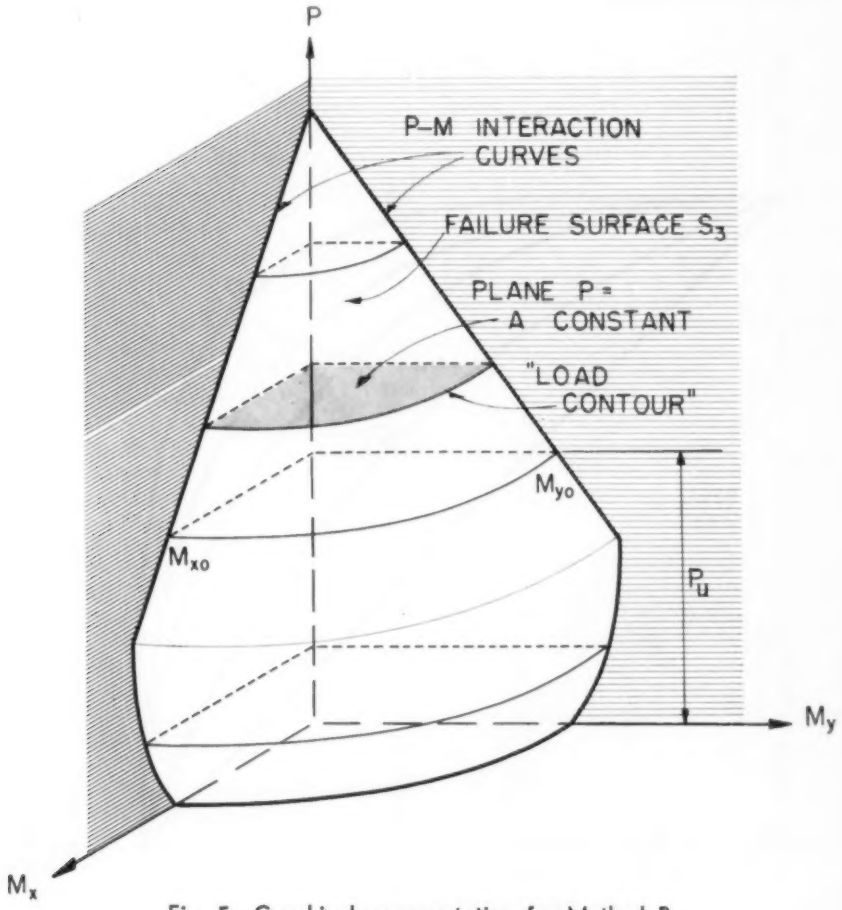

Fig. 5—Graphical representation for Method B

Eq. (5) can be further simplified and transformed into an expression more convenient for design. Dividing all moments in Eq. (5) by P_u results in the following:

$$\left(\frac{y}{y_o}\right)^{\alpha}+\left(\frac{x}{x_o}\right)^{\beta}=1 \qquad (6)$$

Eq. (6) is written in the form of an interaction equation using eccentricities instead of loads or stresses. Assuming $\alpha = \beta$, the shapes of such interaction curves for different values of α are shown in Fig. 6.

For a given case, the design values of P_u, x, and y are generally known, and for a trial section the values of y_o and x_o corresponding to P_u acting with a single eccentricity can be easily determined. Thus verification of the adequacy of the trial section using Eq. (6) becomes a simple procedure.

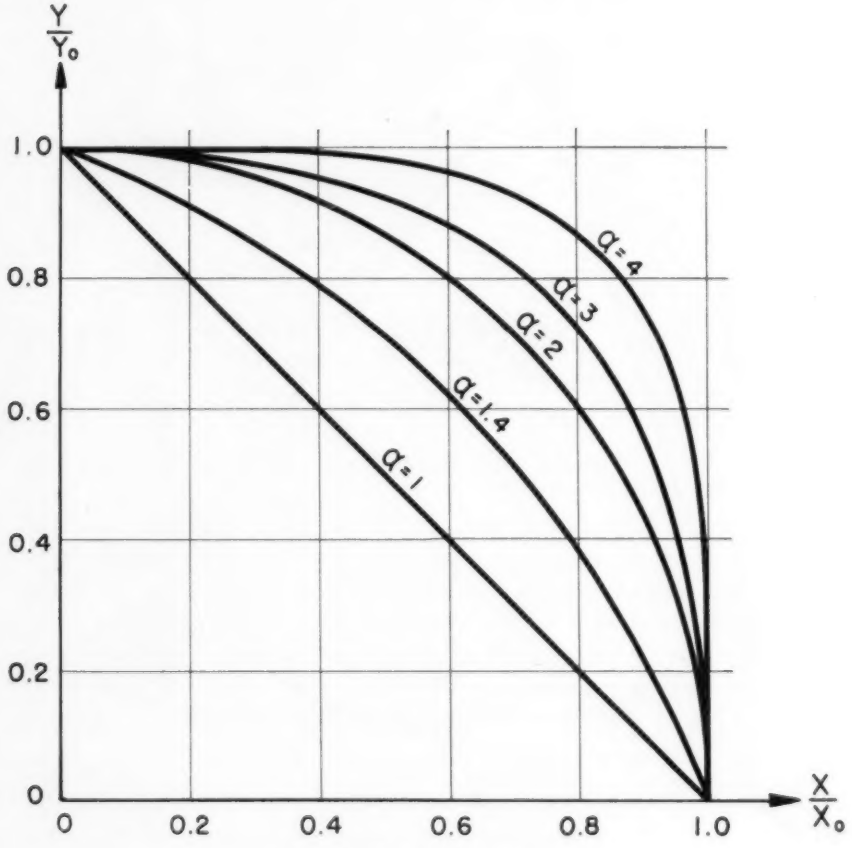

Fig. 6—Interaction curves

ANALYTICAL AND EXPERIMENTAL VERIFICATION

To evaluate the validity of the proposed methods, preliminary calculations and tests were carried out. Values of P_u, M_x, and M_y were calculated for a group of five rectangular columns assuming various positions of neutral axis for each of the columns, and using Jensen's trapezoidal stress-strain law[12] for concrete and the conventional trapezoidal stress-strain law for steel reinforcement. It was found that the strength criteria could be closely approximated by Eq. (6) assuming $\alpha = \beta$. Thus a strength criterion can be defined by Eq. (7).

$$\left(\frac{y}{y_o}\right)^{\alpha} + \left(\frac{x}{x_o}\right)^{\alpha} = 1 \qquad (7)$$

where α is a numerical constant for given column characteristics. The

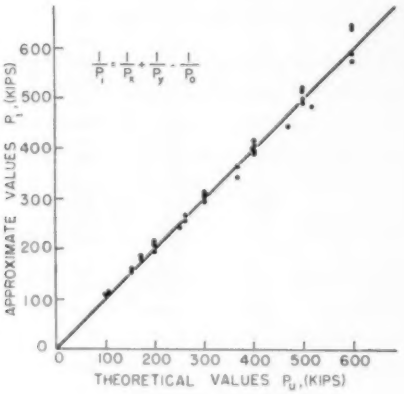

Fig. 7—Comparison of P_i and P_u

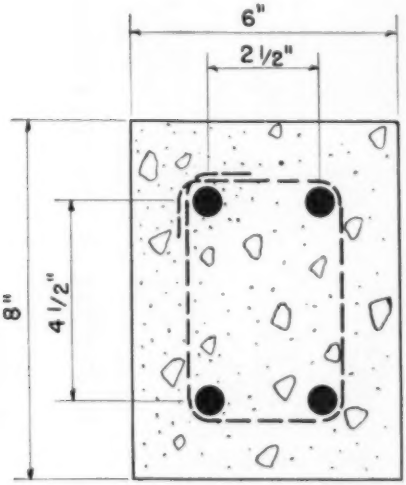

Fig. 8 — Column details. All columns were 4 ft long reinforced with four #5 bars; f_y = 53.5 ksi. Ties were 1/4-in. plain bars spaced at 4 in. on centers. Special loading assemblies were used at ends of columns

TABLE 1 — COLUMN CHARACTERISTICS AND COMPUTED VALUES OF α

Column	Size, in.	Reinforcing steel			Concrete f_c', ksi	Coefficient α†
		Bars*	p, percent	f_y, ksi		
A	15x15	4-#7	1.07	50	3	1.48
B	15x15	8-#7	2.14	50	3	1.35
C	15x15	8-#10	4.49	50	3	1.15
D	12x20	4-#7	1.0	50	3	1.55
E	12x20	8-#7	2.0	50	3	1.45

*Concrete cover for all columns is taken as 2 in. clear for main reinforcing bars.
†See Eq. (7).

TABLE 2 — COMPARISON OF COMPUTED VALUES OF P_i AND P_u

Column (Table 1)	x, in.	y, in.	P_u,* kips	P_x,* kips	P_y,* kips	P_o,* kips	P_i,† kips	P_i/P_u
A	8.1	8.1	105	200	200	795	115	1.09
	5.8	5.8	170	300	300		185	1.09
	4.0	4.0	260	400	400		267	1.03
	2.8	2.8	365	500	500		365	0.03
	1.7	1.7	515	600	600		485	0.94
	11.1	4.2	100	125	395		108	1.08
	8.4	3.5	150	190	470		162	1.08
	7.0	2.7	200	245	515		211	1.06
	4.9	1.8	300	355	585		306	1.02
	3.5	1.3	400	445	640		393	0.98
	2.4	0.9	500	540	690		490	0.98
	1.5	0.6	600	620	725		575	0.96
B	6.8	6.8	170	300	300	915	180	1.06
	4.9	4.9	260	400	400		256	0.98
	3.6	3.6	365	500	500		344	0.94
	2.5	2.5	475	600	600		447	0.94
	10.5	4.8	150	190	405		151	1.01
	7.8	3.6	200	260	500		210	1.05
	5.6	2.5	300	360	600		298	0.99
	4.6	2.1	400	460	650		381	0.95
	3.0	1.2	500	555	765		496	0.99
C	6.8	6.8	250	400	400	1179	241	0.96
	5.3	5.3	300	490	490		310	1.03
	3.9	3.9	400	600	600		404	1.01
	2.9	2.9	500	715	715		513	1.03
	2.1	2.1	600	835	835		645	1.08
	9.1	4.3	250	310	565		242	0.97
	7.2	3.4	300	385	655		305	1.02
	5.1	2.4	400	505	780		415	1.04
	3.8	1.8	500	610	885		531	1.04
	2.8	1.3	600	730	960		640	1.07
E	11.2	3.3	200	250	450	960	193	0.96
	7.5	2.2	300	375	580		299	0.99
	5.4	1.6	400	495	675		407	1.02
	4.1	1.2	500	585	745		502	1.00
	3.0	0.9	600	675	810		590	0.98
	6.8	5.7	200	270	405		194	0.97
	4.5	3.8	300	550	395		303	1.01
	3.2	2.7	400	640	505		400	1.00
	2.4	2.1	500	715	600		492	0.98
	1.8	1.5	600	780	690		592	0.99

*Values computed using Jensen's stress-strain law for concrete.
†Values computed using Eq. (4).

sections used in the preliminary numerical studies and the results of the calculations are shown in Table 1.

Values of P_u, P_o, P_x, and P_y corresponding to selected eccentricities x and y were determined for a group of four columns. Using Eq. (4) values of P_i were calculated for these columns and the results compared with values of P_u computed directly on the basis of stress-strain laws. This comparison is shown in Table 2 and in Fig. 7.

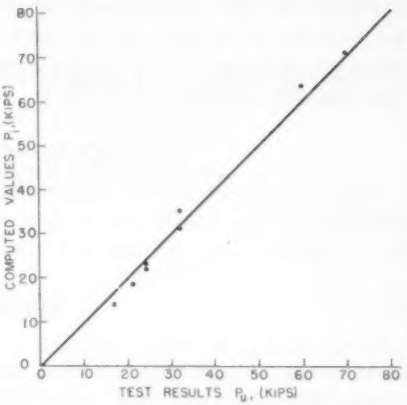

Fig. 9—Comparison of P_i and P_u

Eight columns were tested in the laboratory to determine values of P_x, P_y, and P_u. The details of the columns are shown in Fig. 8 and the test results are shown in Table 3. Values of P_i corresponding to the test values of P_x and P_y were calculated using Eq. (4), and these are compared with the test values in Table 3 and in Fig. 9.

SUMMARY

Two criteria for design of columns subjected to compression combined with biaxial bending were proposed. One, defined by Eq. (7), with calculated values of α varying from 1.15 to 1.55 (see Table 1), was found to provide a good approximation of analytical results. Greater variation in values of α is expected for columns with a wider range of variation in shape and in values of f_c', f_y, and p.

Preliminary verification indicated that ultimate strength P_i predicted by Eq. (4) is in excellent agreement with calculated theoretical values and with test results, the maximum deviation being 9.4 percent, and average deviation being 3.3 percent.

TABLE 3 — COMPARISON OF TEST RESULTS WITH COMPUTED VALUES

Column*	f_c', ksi	x, in.	y, in.	Test P_u	Computed				P_i/P_u
					P_x†	P_y†	P_o‡	P_i§	
B-1	3.7	6	0	24	23.6	—	—	23.6	0.98
B-2	3.9	3	0	60	63.8	—	—	63.8	1.06
B-3	3.7	0	4	70	—	71.3	—	71.3	1.02
B-4	4.6	0	8	32	—	31.1	—	31.1	0.97
B-5	3.2	3	4	32	55.5	65.2	191.4	35.3	1.10
B-6	3.7	6	8	17	23.6	30.2	210.8	14.2	0.84
B-7	3.5	6	4	21	23.4	69.0	203.3	18.8	0.90
B-8	3.6	3	8	24	59.5	28.3	205.5	21.6	0.89

*f_y = 52.3 ksi for all columns.
†Values computed using Eq. (A9) or (A11) of the appendix in ACI Building Code ACI 318-56.
‡Values computed using Eq. (A6) of the ACI Building Code.
§Values computed using Eq. (4).

Based on the preliminary studies outlined above, Eq. (4) appears to provide a simple, direct, and accurate approximation of ultimate strength of a reinforced concrete column subjected to axial compression and biaxial bending.

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Title No. 57-24

Corrosion of Prestressed Wire in Concrete

By G. E. MONFORE and G. J. VERBECK

After a brief review of the various types of corrosion, the few corrosion failures which have occurred in prestressed wire in concrete are discussed. The failure in the Regina, Sask., Canada, water supply line is examined in detail. A summary of the laboratory investigations of corrosion of prestressed wire in concrete that have been reported in the literature is followed by an account of the studies carried out in the laboratories of the Portland Cement Association. These latter studies included tests of the effects on corrosion of such factors as type of cement, type of wire, wire stress, addition of calcium chloride, curing conditions, storage conditions, and voids in the concrete adjacent to the wires. Many of the tests were evaluated by determination of the reduction in tensile strength of the wires. Field and laboratory observations indicate clearly that calcium chloride in concrete may lead to serious corrosion of prestressing steel. Because of this hazard, it is recommended that calcium chloride not be used in prestressed concrete.

■ CORROSION IS USUALLY DEFINED as the destruction of a metal by chemical or electrochemical reaction with its environment.[1] Although corrosion has been a problem since metals, especially iron and steel, were first used as structural materials, a complete knowledge of the process is still lacking. Many volumes have been written about the subject and the periodical literature contains thousands of references, but most of this information is of an empirical nature. Much work is being done on the mechanism of corrosion, and in the field of homogeneous alloys some basic principles have been discovered.[2] In general, however, the performance of a new alloy or the performance of a familiar alloy in a new environment can not be completely predicted at present. What will happen to a particular metal in a particular environment must be learned by testing that particular metal under the particular conditions.

Various types of corrosion have been recognized and are classified in several ways. One way is to classify corrosion according to the effect on the metal, as general or localized.[3]

General corrosion

The uniform diminution of a metal by chemical attack is designated general corrosion. Such attack requires that ideally every atom in the exposed surface be equally sensitive to the corrosive medium and that the medium have uniform access to all surface atoms. This abstract case probably never actually occurs, but it is approached in the action of acids on some metals. For example, when iron is immersed in hydrochloric acid, the iron is oxidized, i.e., loses electrons, and hydrogen ion is reduced, i.e., gains electrons.

$$Fe + 2H^{+} \rightarrow Fe^{++} + H_2$$

This reaction proceeds spontaneously to the right with nearly uniform solution of the metal until it is entirely consumed.

Localized corrosion

As the surfaces of metals and corrosive environments are seldom homogeneous, the attack will usually not be uniform but will be localized. For example, if iron is placed in neutral sodium chloride solution, there will be localized areas acting as anodes where the iron is oxidized and other areas acting as cathodes where hydrogen ion is reduced.

$$\text{Anode:} \quad Fe \rightarrow Fe^{++} + 2e$$
$$\text{Cathode:} \quad 2H^{+} + 2e \rightarrow H_2$$

This action is called electrochemical in contrast to the designation chemical as applied to uniform attack. In electrochemical attack the anodic and cathodic areas are separated by finite distances and corrosion is accompanied by electron flow within the metal.

The rate of electrochemical attack may be influenced by the products which are formed either at the anodic or cathodic areas. If these products are soluble, a continuing reaction is favored. If the products are insoluble and tend to form tightly bound films, the reaction may be completely stifled. For instance, copper exposed to the atmosphere slowly develops a thin coating which is very resistant to further attack.

Pitting corrosion — If the localized attack is independent of the crystal structure, the corrosion is called pitting. Pits are formed in the surface of the metal at randomly distributed locations. The rusting of iron in a humid atmosphere is a well known example of pitting corrosion.

Intercrystalline corrosion — Since alloys are composed of a large number of crystals with correspondingly large intercrystalline surfaces, it is not surprising to find that these intercrystalline boundaries are subject to corrosive attack under some conditions. An alloy may be initially a homogeneous solid solution, but may develop concentrations of one component at the crystal boundaries due to subsequent heat treatment. These differences in composition between crystals and crystal boundaries may cause anodic and cathodic areas in the presence of the appropriate

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environment. Such highly localized corrosive attack is called intercrystalline corrosion. Although this type of corrosion occurs without the aid of tensile stress, such stress may greatly accelerate the rate of attack.

Stress corrosion cracking — Another type of highly localized corrosion is that called stress corrosion cracking and is defined as the cracking of an alloy that may result from the combined action of corrosion and static tensile stress, which may be either residual or externally applied. The effect on the metal may be an intercrystalline crack, the same as occurs in intercrystalline corrosion. Stress corrosion cracking, however, does not occur in the absence of stress, whereas intercrystalline corrosion does. Unlike intercrystalline corrosion, the cracking in stress corrosion may also be transcrystalline in form, i.e., the cracks may pass right through the individual crystals.

Although the mechanism of stress corrosion cracking is certainly complex and imperfectly understood, the fact that this type of attack occurs only in alloys indicates that the internal metallurgical structure, which is influenced by composition, heat treatment, and mechanical processing, must be a major factor. From studies of homogeneous alloys of copper-gold and silver-gold, Graf found that "stress corrosion cracking susceptibility is caused by alloy components that are more noble than the principal metallic component of the alloy."[2] Parkins presented evidence that the susceptibility of mild steels to stress corrosion cracking in boiling calcium-ammonium nitrate solutions is related to the presence of carbide particles within the crystal boundaries.[4] Steels which were initially resistant to stress corrosion cracking became susceptible after prolonged annealing. Apparently carbide migrated to the crystal boundaries during the annealing process.

The literature contains many other references to stress corrosion cracking but nearly all are of an empirical nature. Pollard produced cracking of high carbon bridge-cable wires by immersion in 0.01 normal solutions of ammonium or sodium nitrate.[5] Oil-tempered high carbon wire was found by Everling to be more susceptible to cracking in hot ammonium nitrate solutions than a similar hard drawn wire.[6] Stress corrosion cracking (caustic embrittlement) has occurred at riveted joints

in steel boilers where contact between metal surfaces permitted the accumulation of highly concentrated caustic solutions.[7]

The ductility of steel has been shown to be reduced as the hydrogen content is increased.[8] This phenomenon is not directly a factor in stress corrosion cracking, but Evans has suggested that atomic hydrogen formed at the tip of a crack by corrosive action might be a factor in the cracking rate.[9]

Other types of localized corrosion are recognized, such as corrosion fatigue, selective corrosion, and galvanic contacts,[3] but they do not seem pertinent to the present discussion.

REPORTED FAILURES OF PRESTRESSED CONCRETE STRUCTURES

Hundreds of linear prestressed concrete structures, such as bridges and building frames; hundreds of circular prestressed concrete tanks; and hundreds of miles of prestressed concrete pipe have been built in the past several years at locations including all of the continents. The number of reported structural failures in this vast amount of prestressed concrete work is exceedingly small. The conclusion can thus be drawn that corrosion of prestressed wire in concrete is not a common problem. In fact, experience proves that concrete normally provides excellent protection against corrosion to a material which is naturally nonresistant to outdoor weathering. Nevertheless the prestressed concrete industry should guard against the occurrence of any failure, and hence the failures which have occurred must be studied carefully for clues as to why they occurred.

Miscellaneous failures

Bouvy cites five incidents, which occurred in Holland, Germany, and Austria, involving the failure of "hot-rolled tempered" steel wire.[10] In each case ruptures of the wire occurred either while the wire was still in the coils as received from the manufacturer or so soon after tensioning that the concrete had not yet been placed. These failures were attributed to a high susceptibility to stress corrosion of hot-rolled tempered wire, coiling of the wire in small diameter rolls with attendant high bending stresses, and storage of the coils in corrosive environments. Although these cases did not involve the corrosion of wire in concrete, they do point out that the properties of the wire, which are a result of composition and manufacturing procedures, determine the susceptibility to certain localized corrosion.

The corrosion of post-tensioned cables in the Richmond reservoir in California has been reported.[11] The 18-strand cables, which were contained in 1½-in. conduits, were tensioned after the concrete had hardened. The conduits were not grouted immediately because excess friction on tensioning indicated that the cables should be retensioned

again at a later date. When the retensioning was begun after a delay of 5 months, many cables were found to have strands which were broken due to corrosion. Although this again is not a case of corrosion in concrete, it emphasizes the necessity of properly protecting high strength steel wire.

Corrosion in the prestressing cables around a tank in England was mentioned by Hill.[12] The corrosion occurred at the base of the wall where water had penetrated by capillary action between the shotcrete coating and the parent concrete. The bond between the shotcrete and the parent concrete was apparently poor.

Failures of the wires in a circular tank in Algeria was described by Bouvy as occurring some time after prestressing.[10] The tank was built by the Preload system and "here the cold-drawn wire used was at its limit of cold hardening, and could not stand up to the reduction in diameter caused by passing through the die of the prestressing machine. This, together with notching and stress-corrosion effects, caused the failure."

Corrosion failures in the wires of a dome roof of the sprinkling filter unit at the Regina, Sask., Canada, sewage disposal plant were reported as occurring within 5 months after construction. Calcium chloride, perhaps as much as 5 percent by weight of the cement, was used in the shotcrete mortar.

The failure of wires in several prestressed concrete pipe projects were reported by Evans.[13] Wire failures occurred in the pipes in a pipe line in Belgium about 6 weeks after the pipes were laid. The trench was still open but was filled with aggressive ground water. "Investigation showed that fracture took place in the area of contact between the pipe and the ground and generally only at one end of the pipe. It was found that the concrete cover to the reinforcement was fairly porous and its thickness at the place of failure was much less than the specified requirement owing to a defect in the mold at this end. No trace of corrosion was noted in the center portion of the pipes where the cover was at least 1 in." Failure of the wires in some 35-in. prestressed pipes in France occurred after 6 months of storage on damp ground. High-strength portland cement was used in the core and portland blast-furnace cement was used in the cover. The failures were apparently limited to the ends of the pipes which were in contact with the damp ground. The only detail given concerning a pipe failure in North Africa was that different cements were used in the core and cover. Only a reference was given concerning a failure in Brazil.

Everling reported failures in two prestressed concrete pipe lines which occurred within approximately a year of their construction.[6] Oil-tempered wire was used in both projects.

Regina pipe line

The fabrication of 36-in. diameter prestressed concrete pipe for a 35 mile water supply line for the city of Regina, Sask., Canada, was begun in March, 1952.[14] In mid November, 1952, a serious leak developed in one 16-ft section of pipe during a pressure test of installed pipe. About 5 miles of pipe had been installed at that time and pipe for perhaps 8 miles more had been fabricated and was being stored in the plant yard. Examination of the section of pipe, which was 4½ months old at the time, revealed that the circumferential prestressed wires had failed due to corrosion. There were hundreds of breaks in this one 16-ft section of pipe. Further examination showed corrosion of the wires in most of the other sections of pipe, both installed and stored. The production of pipe was stopped Dec. 23, 1952.

The pipe was built in three steps. The core, which was 2 to 2½ in. thick and contained 24 longitudinal wires 0.162 in. in diameter stressed to 140,000 psi, was fabricated first using shotcrete mortar containing Type I portland cement. This core was cured with water spray at 140 F for at least 10 hr. The second step consisted in winding wire under stress on the hardened core. This was accomplished by drawing a 0.162-in. wire through a 0.142-in. die with power being furnished by the rotating pipe. The third step in manufacture consisted in placing a mortar cover over the circumferential wires. The cover coat was applied pneumatically in the same manner as employed for the core, but a sulfate resisting cement was used in the mortar. After the cover was applied to a nominal thickness of ¾ in., the pipe was cured with water at 140 F for another 10 hr.

A photograph of a 16 x 24-in. section from one of the pipes, which is shown in Fig. 1, illustrates the locations of the longitudinal and circumferential wires. A similar size section from the pipe which failed during the pressure test was broken apart for examination. Fig. 2 shows the surface of the core after the cover was removed, and Fig. 3 shows the inner side of the removed cover. Some of the fractured wires are still

Fig. 1 — Section of Regina pipe showing circumferential and longitudinal wires

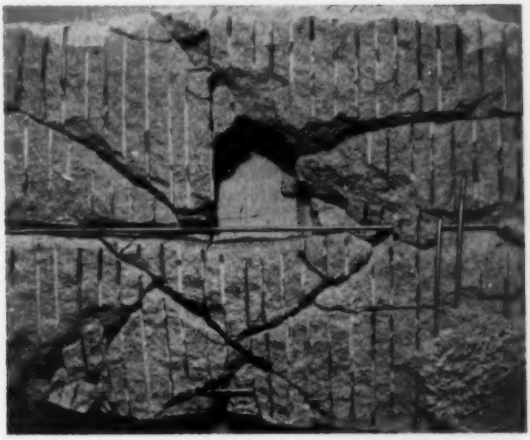

Fig. 2—Surface of core of Regnia pipe after cover was removed

in place, and other pieces of wire which were removed from this section are shown in Fig. 4. There were about 30 corrosion breaks in the wire in this one 16 x 24-in. section of pipe. Enlarged views of several corroded wires taken from pipes are reproduced in Fig. 5 and illustrate the severe pitting and axial splits which were typical of the corrosive attack.

Many experts were called in to assist in the investigations of the failure of the Regina pipe. The following observations are taken from the reports of these investigators.

The shotcrete mortar used in the first 84 pipes manufactured consisted of one part cement to approximately 2.5 parts sand and 0.42 part water. The mortar for the other 4400 pipes was the same except that it contained calcium chloride in concentrations from about 1½ percent to over 3 percent by weight of cement. After the failure occurred in the pipe line, many sections of pipes, both those which had been installed and others still in storage, were broken open and examined. No corrosion was found in the wires of any of 29 pipes selected for examination

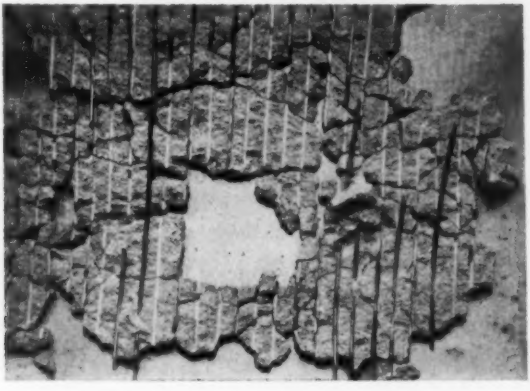

Fig. 3 — Inner surface of cover from Regina pipe

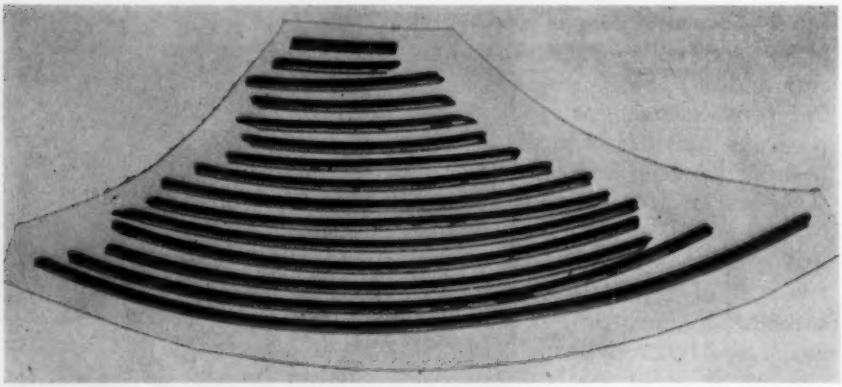

Fig. 4—Fragments of wire from section of Regina pipe

from the 84 pipes manufactured without calcium chloride. One of these pipes was completely demolished so that every inch of wire was exposed. No corrosion was found. About 58 pipes made with mortar containing calcium chloride were examined and corrosion was found in every case. The conclusion that calcium chloride was the primary source of corrosion seems inescapable. Opinions varied, however, as to what other factors might have been involved.

One investigator attributed the severe localized corrosion to calcium chloride plus electrical potentials set up between the core and the cover.

The water which was used for curing the pipes was recirculated and was thus used over and over again. This procedure caused an increase in the salt content. A concentration of 2.0 g of calcium sulfate and 1.4 g of calcium chloride per liter was determined by one analysis of the curing water.

The cold-drawn 0.162-in. steel wire was purchased under a specification requiring a minimum ultimate tensile strength of 210,000 psi and a limit of 1 percent elongation at a stress of 180,000 psi. Subsequent tests indicated that the wire which was furnished for the project easily met these requirements. The carbon content of the wire was between 0.6 and 0.7 percent.

The 0.162-in. wire was used as received for the longitudinal reinforcement. There was some criticism of the wire drawing procedure used in wrapping the circumferential wires. Apparently water-cooled dies are commonly used by wire manufacturers in drawing high carbon wire to avoid excessive heat arising from friction. Also, and for the same reason, the wire is usually lubricated. A high temperature rise in the drawing process tends to decrease the ductility of the metal near the surface and to induce circumferential tensile stresses in the surface. Since the die used at the Regina plant was reportedly used

without cooling or lubricant, it is possible that the susceptibility of the circumferential wire to splitting by corrosion was thus increased. After metallographic examination of several pieces of wires from failed pipes, one investigator stated that all fractures which he observed were associated with longitudinal splits.

Examination of many corroded pipes established unquestionably that the circumferential wires were corroded more severely than the longitudinal wires. This could be associated with a greater inherent susceptibility of the circumferential wire to splitting under corrosive action

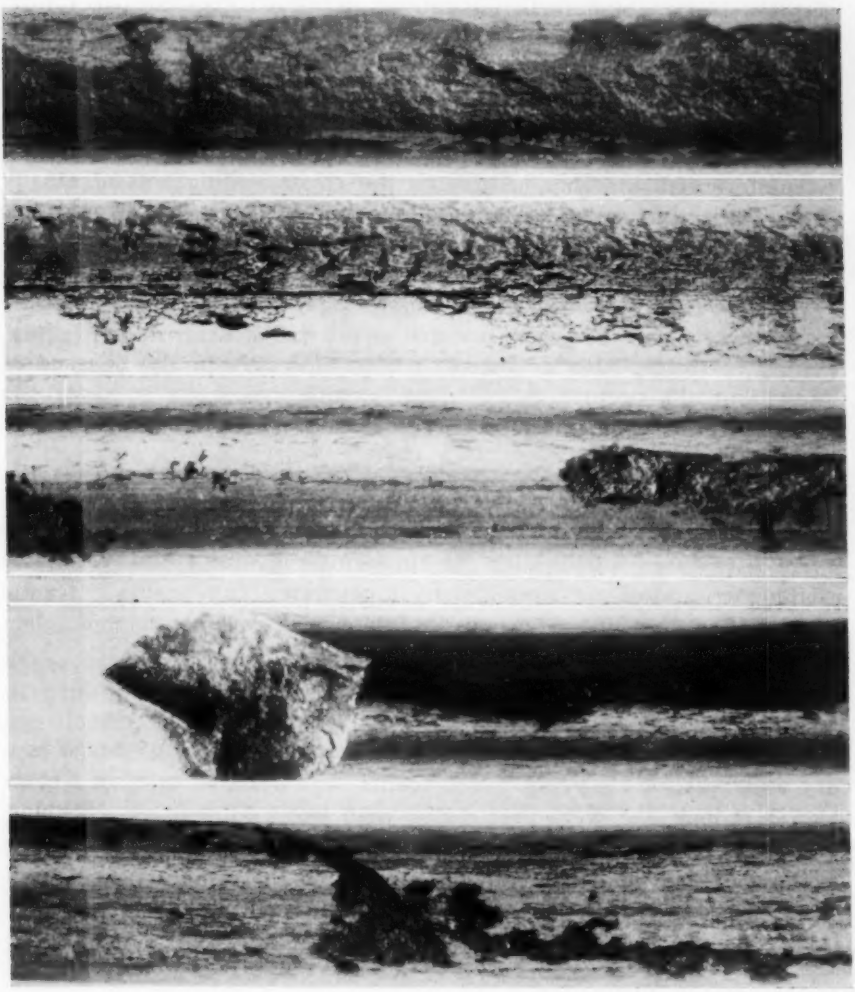

Fig. 5—Enlarged views of wires from Regina pipe

as just discussed. There were also other factors, which tended to corrode the circumferential wires more than the longitudinal wires. The longitudinal wires were stretched in a frame and were then covered with shotcrete. This procedure favored a good bonding of the mortar all around the wire. By contrast, the circumferential wires were wound on a hardened rough mortar surface and were then covered with shotcrete. This method tended to leave voids underneath the wires. Examination of pipes by several investigators showed that the corrosion of the circumferential wires was indeed much greater on the core side. Another factor which favored corrosion of the circumferential wire resulted from a manufacturing procedure. The collapsible steel form on which the core was placed was not removed until after the pipe was completed and cured for the second time. The tendency of the core to be compressed slightly upon removal of the form would also create a tendency toward separation of the core from the cover. Such separation would present a path for water to seep into the pipe along the circumferential windings. Although the cover coating was nominally ¾ in. thick, thicknesses of only ⅛ in. were observed. This would also put the circumferential wires in an unfavorable position as compared to the longitudinal wires.

The fact that the wires in the Regina pipe were under high tensile stresses suggested the possibility that stress was a factor in the failure. Pitting corrosion, such as shown in Fig. 5, occurs in the absence of stress, but the occurrence of many longitudinal splits suggests the operation of internal circumferential tensile stress.

LABORATORY INVESTIGATIONS OF CORROSION OF PRESTRESSING WIRE IN CONCRETE

The literature contains little information on laboratory investigations of the corrosion of prestressed wire in concrete. This probably results from the fact that under normal conditions steel does not corrode when embedded in concrete.

Veits

Veits reported tests on prestressed concrete beams after storage in a dry basement for 2½ years.[15] The beams were 2 x 4 in. in cross section by 7½ ft long and were prestressed with wires 0.079 in. in diameter. The concrete was cast on a vibrating table and was steam cured. Beams which contained no calcium chloride exceeded the computed load limit when tested in flexure. Companion specimens, which contained 2 percent calcium chloride by weight of cement, failed under a very light load, and the wires were found to be corroded more than half way through in some places.

Evans

An extensive laboratory investigation, including both prestressed pipes and prisms, has been reported by Evans.[13] The pipes were 7¼ in. I.D. and 13½ in. O.D. Cores were cast by centrifugation and steam cured for 48 hr at 160 F. Wires were then wound around the cores under stress, cover coats were applied, and the pipes were again steam cured for 48 hr at 160 F. Portland cement, with and without 2 percent calcium chloride, was used for both cores and covers of some pipes; in a similar manner sulfate resisting cement and combinations of "portland cement" and "sulfate resisting cement," with and without 2 percent calcium chloride were used in other pipes. After 6 to 8 weeks' storage the pipes were broken open and the wires were examined. In every case in which calcium chloride was used in the core, in the cover, or in both, there was localized corrosion of the wires. The type of cement or combinations of cements apparently had little if any effect. There was no corrosion in any pipes in which calcium chloride was not used. A few pipes, which contained no calcium chloride but in which the type of cement in core and cover was varied were cured in hot water at 160 F, and another similar group was cured at atmospheric temperature. There was no corrosion in any of these pipes.

In another part of the investigation a large number of 2 x 2 x 30-in. pretensioned prestressed concrete prisms were cast in which portland and sulfate resisting cements were used, some without calcium chloride, some with 2 percent, and some with 4 percent calcium chloride by weight of cement. The wire was 0.128 in. in diameter and was located on the axes of the prisms. Prisms were made with portland cement only, with sulfate resisting cement only, and with combinations of the two. In the composite prisms the wires lay on the division between the two types of cement. Results of prisms steam cured at 180 F for 8 hr and examined after about 2 months verified results obtained from the pipe tests. Corrosion occurred only when calcium chloride was present, and type of cement or combinations of two cements had no apparent effect. There was no obvious difference between the corrosion of wires under a load of 140,000 psi and the corrosion of wires under zero load. Corrosion of wires in concrete containing calcium chloride was found to be less severe after curing in water at 180 F than after curing in steam at 180 F. Wires embedded in concrete containing calcium chloride and cured for 6 months at atmospheric temperature were found to have patches of discoloration indicative of slight corrosion.

Portland Cement Association

The investigations of corrosion of prestressed wire in concrete which are being carried out in the laboratories of the Portland Cement Association were initiated by the failure of the Regina pipe, but the purpose of the studies has not been directed especially to an explanation of

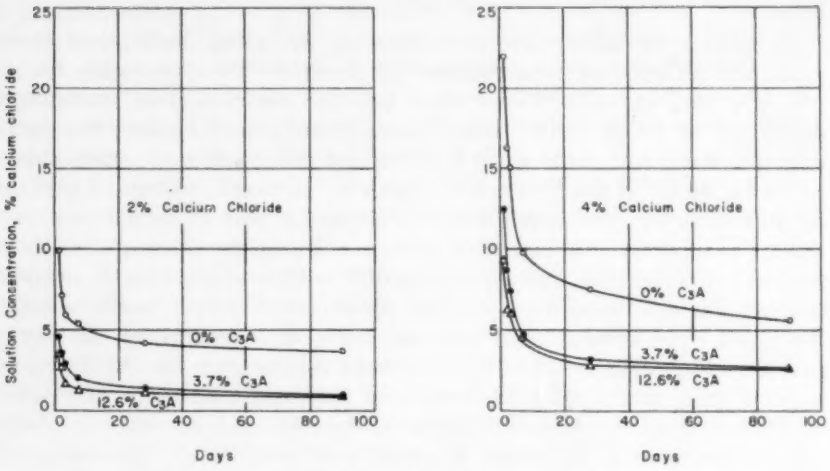

Fig. 6—Soluble chloride in cement paste

that failure. The aim has been rather to gain a better basic understanding of the factors which affect the corrosion of prestressing steel embedded in concrete.

Susceptibility to stress corrosion cracking — The susceptibility to stress corrosion cracking of several 0.162-in. high strength steel wires was studied by immersing the wires in ammonium nitrate solutions. Although nitrate solutions are commonly used in studies of the cracking tendencies of steel wires, the results may not be of value in predicting the behavior of the wires in another environment. The wires were subjected to four external loading conditions: unloaded, tensile load, torsional load, and bending load. Fractures, which occurred only in the loaded wires, were due primarily to the formation of cracks, but there was also some pitting. Corrosion failures in the tensile specimens were the most reproducible and those in the bent specimens were the most erratic. Failures in the torsional specimens were mainly due to longitudinal cracks. Of the wires tested, an oil-tempered wire was by far the most susceptible to stress corrosion cracking. A sample of the hard-drawn wire used in the Regina pipe was apparently somewhat more susceptible to longitudinal cracking than a hard-drawn wire from another manufacturer.

Calcium chloride in cement paste — Since calcium chloride has been a primary factor in the corrosion of steel wire in prestressed concrete, the amount of soluble chloride which is available at different ages is probably important. To study this relation, 2-in. neat cement cubes were made from three different cements with 2 and 4 percent calcium chloride by weight of cement added. Three cubes were pulverized at

TABLE 1 — CHLORIDES IN WATER-CEMENT SLURRIES

Percent $CaCl_2$, by weight of cement, in solution after the following number of days					
0	1	2	3	7	28
0.25	0.12	0.08	0.04	0.03	0.03
0.50	0.26	0.16	0.12	0.08	0.07
1.00	0.51	0.34	0.25	0.17	0.15
2.00	1.13	0.77	0.61	0.41	0.37
4.00	2.74	2.15	1.96	1.45	1.10

each of the various test ages and water was passed through a bed of the pulverized cement. The first 5 ml passing through was collected and the concentration of calcium chloride was determined. Results are shown in Fig. 6. The cements which were used in the study contained 0 (3.1 percent Al_2O_3), 3.7 and 12.6 percent calculated tricalcium aluminate. The most striking aspect of the results was the rapid reduction in the readily soluble chloride which occurred in the first 10 days. Another important conclusion was that an increase of the tricalcium aluminate content of the cement from 0 to 3.7 percent reduced the soluble chloride by over 50 percent. Increase of tricalcium aluminate content from 3.7 to 12.6 percent caused an additional reduction of soluble chloride of about 25 percent at early ages but less at later ages.

Additional information on the amounts of soluble chloride in water-cement systems was obtained by reacting cement with water containing various quantities of calcium chloride. Slurries, which consisted of 3 parts water, 1 part Type I cement (12 percent calculated C_3A), and percentages of calcium chloride by weight of cement of 0.25, 0.50, 1, 2, and 4, were agitated continuously for various periods. At the end of each period the quantities of calcium chloride remaining in solution were determined by chemical analyses. The results, which are listed in Table 1, show that the amount of chloride ion unreacted and remaining in solution decreased only slightly after 7 days. Also, regardless of the initial amount of calcium chloride, appreciable quantities remained in solution after 28 days.

Corrosion in solutions — Hard-drawn wires, some 0.162 and some 0.142 in. in diameter, were immersed in solutions of various combinations and concentrations of calcium chloride, calcium hydroxide, sodium chloride, and sodium hydroxide while under an applied tensile load of 120,000 psi. Although some of the solutions caused rapid localized corrosion initially, the rate of corrosion decreased rapidly. To accelerate the corrosion rate, external voltages were applied; wire failures were then obtained under some conditions in a few days. The rate of corrosion depended upon the composition of the solution and upon the direction and magnitude of the current. Under certain testing conditions, the addition of calcium hydroxide to pure water was found to decrease the corrosion

rate, while the addition of calcium hydroxide or sodium hydroxide to a solution containing chloride ions increased the rate of corrosion. Thus, although calcium hydroxide normally acted as a corrosion inhibitor in the absence of chloride ions, in the presence of chloride ions both calcium and sodium hydroxide acted as accelerators, the degree of acceleration increasing with increasing alkalinity.

Effect of molds — Potentials between wires used for prestressed concrete and several other metals which are likely to be used as molds in laboratory investigations were measured when the metals were embedded in mortars made with Type I cement containing 4 percent calcium chloride. In these tests, as well as in all other tests in the Portland Cement Association studies, the calcium chloride was dissolved in the mixing water and the percentage was that by weight of cement. Potentials were measured with a deflection type instrument to avoid the application of any external voltage to the system. The absolute magnitudes of the voltages measured were not significant since they depended, not only on elapsed time, but also on the resistance of the measuring instrument. However, relative values did have some significance. Under certain measuring conditions, the voltage between wire and brass was +160 mv, between wire and stainless steel +30 mv, and between wire and ungalvanized sheet metal −30 mv. Positive voltages are used to indicate the direction of current which would result when iron ions are going into solution at the wire. Thus the potential between the wire and brass was such as to corrode the wire, that between the wire and stainless steel was less corrosive to the wire, and that between the wire and ungalvanized sheet steel tended to corrode the sheet steel instead of the wire.

Corrosion tests were then made by embedding wires in mortars contained in molds made from brass, stainless steel, ungalvanized steel, and wood. Four percent calcium chloride was used in the mortar. The specimens were cured at 73 F for 1 or 3 days, and were then broken open and examined. The results were in the same order indicated by the potential measurements. Wires cured in brass molds showed the greatest amount of corrosion, those cured in stainless steel showed considerably less corrosion, and those in the ungalvanized steel molds were practically unaffected. The ungalvanized molds, however, were considerably corroded. Wires cured in wood molds showed appreciable corrosion after 1 day.

The results of these studies are of interest primarily in laboratory investigations, and point out that corrosion of prestressed wire during the period that the specimen is in the mold may be considerably influenced by the composition of the mold. More generally the results indicate that any metal in contact with the concrete may affect the corrosion of the wire. Perhaps it should be emphasized that these results apply when the concrete contains soluble chloride, and does

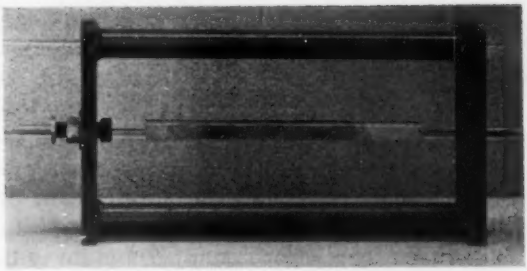

Fig. 7—1 x 1 x 18-in. mortar specimen in stress rack

not imply any adverse effects from the use of metal molds with concrete free of soluble chloride.

Corrosion in 1 x 1 x 18-in. mortar specimens—Certain factors which influence the corrosion of prestressed wire were investigated by embedding wires in 1 x 1 x 18-in. mortar prisms. Fifty-four specimens were fabricated in these preliminary studies which included effects of variations in both the properties of the mortar and the properties of the wire.

Externally applied loads were maintained at 140,000 psi by means of heavy steel frames as shown in Fig. 7. The wire loads were checked periodically with a calibrated hydraulic jack and adjusted if necessary. This procedure had the advantage of maintaining a known and constant external load on the wire, but it had the disadvantage of not applying compressive load to the mortar. The mortar in some specimens developed tensile cracks due to shrinkage of the mortar and creep in the wire.

The specimens were stored in a room at 73 F and 50 percent relative humidity. The central portions of the specimens, however, were wrapped in wet towels. Most of the prisms were examined after 28 days' storage. The first step in examination consisted in sawing lengthwise slots along two opposite sides of the specimens. The prisms could then be split open, the wires removed, and both wire and adjacent mortar examined. Although in some cases degrees of severity of corrosion were assigned, such distinction on the basis of visual observation is difficult. The natural inclination is to associate more severe corrosion with greater area, whereas the important factor is depth of penetration which is not readily apparent. These studies were accordingly largely qualitative.

The mortar consisted of 1 part cement, 2.5 parts sand, and 0.4 part water. Most mixes also contained 4 percent calcium chloride by weight of cement. Corrosion occurred in all wires embedded in mortar containing 4 percent calcium chloride regardless of the type of cement used. Some specimens were made with Type I cement, some with Type V cement, some with a special cement which contained no calculated tricalcium aluminate (3.1 percent Al_2O_3), and others were composite specimens with Type I cement in the mortar on one side of

the wire and Type V cement on the other side. The only noteworthy difference in the effect of these various mortars was that certain wires developed longitudinal cracks more readily in the presence of the Type V cement than in the presence of the Type I cement.

That calcium chloride was the prime factor in the corrosion was proved by making some specimens without calcium chloride. There was no corrosion of wires in mortars which contained no calcium chloride whether Type I cement, Type V cement, composite of Type I and V cements, or a zero tricalcium aluminate cement was used. A few specimens in which calcium chloride was replaced by an equivalent amount of sodium chloride exhibited apparently about the same amount of corrosion as those with calcium chloride.

Most of the specimens were cured at atmospheric pressure in steam at 180 F for 12 hr, but some were cured at 73 F for either 1 or 3 days. Corrosion in specimens cured at room temperature and stored for 28 days under wet towels appeared to be about the same as the corrosion in comparable steam cured specimens.

The corrosion of a particular hard-drawn wire was apparently somewhat greater when embedded in a mortar with a water-cement ratio of 0.4 than when embedded in a mortar with a water-cement ratio of 0.6. This effect of water-cement ratio probably resulted largely from the placability of the mortars. The mortar with the 0.4 water-cement ratio was quite dry and left more numerous and larger voids next to the wire than did the wetter mix.

Five different wires were used in the studies. Three of the wires, which will be designated as the "A" wires were furnished by one manufacturer and were from the same stock. Part of the wire was cold-drawn by the manufacturer to a diameter of 0.142 in. Half of this wire, identified as A-as-drawn, was sent to our laboratory in the as-drawn condition, and the other half, identified as A-SR, was stress relieved before being sent. Some of Wire A was also furnished in a diameter of 0.162 in., and the final draw from 0.162 to 0.142 in. was performed in our laboratory with an uncooled die and unlubricated wire, this wire being identified as A-PCA. The fourth and fifth wires were from the stock used in the Regina pipe. Some of the wire was drawn at the project from 0.162 to 0.142 in., and some designated as Regina-PCA, was drawn in our laboratory from a diameter of 0.162 to 0.142 in. using the procedure just described.

All wires corroded when they were embedded in mortars containing 4 percent calcium chloride and were stored for 28 days under wet towels.

A few specimens were stored under wet towels until the wires failed by corrosion. Fractures occurred when the cross-sectional area of the wires had been reduced by about 40 percent largely by pitting corrosion. One specimen containing Wire A-stress-relieved failed after 44 days

of storage; one Regina-PCA wire failed after 26 days, but another similar wire did not fail until stored for 92 days; one Wire A-PCA failed after 103 days; and one Wire A-as-drawn did not fail in 162 days.

The procedure of drawing unlubricated wire through an uncooled die increases the susceptibility of the wire to development of longitudinal cracks. Neither Wire A-stress-relieved nor Wire A-as-drawn developed longitudinal cracks during 28 days or more embedded in mortars containing calcium chloride, but both Wire A-PCA and Regina-PCA wire develop longitudinal cracks under the same conditions.

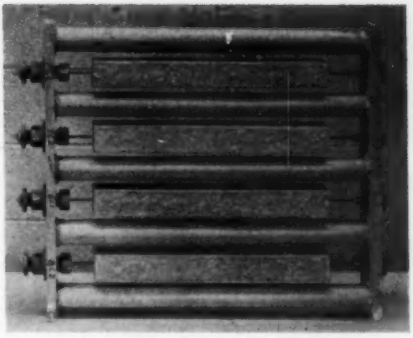

Fig. 8 — 2 x 2 x 18-in. concrete specimens in stress rack

Most of the wires were loaded to 140,000 psi during corrosion exposure, but a few were tested at zero load. These unloaded wires apparently corroded about as much as comparable loaded wires during 28 days' storage under wet towels. The conclusion can not be drawn, however, that the external load would not ultimately affect the rate of penetration.

Corrosion in 2 x 2 x 18-in concrete specimens — In another study of corrosion, stressed wires were embedded along the axes of 2 x 2 x 18-in. concrete prisms. A total of 48 specimens were cast. Wire loads were maintained constant by means of heavy steel frames, Fig. 8, in the same manner as was used for the 1 x 1 x 18-in. mortar specimens.

Type I and V cements were used in the sand and gravel concrete. Maximum size aggregate was ⅜ in. Cement contents were selected to produce 28-day strengths of 6000 and 8000 psi for both types of cement. Calcium chloride was added to the mix water in amounts by weight of cement of 0, 2, and 4 percent. As the slump of all mixes was essentially zero, the specimens were vibrated on a vibrating table. The prisms were sealed in plastic film and cured for 28 days at 73 F. Half of the specimens were stored in a room where the temperature was controlled at 73 F and the relative humidity was maintained at 50 percent; the other specimens were stored in the same room, but the central portions were covered with wet towels. The wires in four of the specimens ruptured during the storage period of 2 years. As might be expected, these ruptures occurred when the wires were being retensioned. The other 44 specimens were broken open, examined, and the tensile strength of the wires determined at the end of the 2 years. Detailed results of the relative tensile strengths are listed in Table 2.

Greater corrosion might be expected in specimens stored under wet towels than in those stored dry, and the results in Table 2 indicate that

TABLE 2 — CORROSION IN 2 x 2 x 18-IN. CONCRETE SPECIMENS

Calcium chloride, percent	Type cement	Cement content, sack per cu yd	W/C, gal. per sack	Reduction in tensile strength, percent			
				Stress-relieved wire		As-drawn wire	
				Dry storage	Wet storage	Dry storage	Wet storage
0	I	4¾	6	0	0	0	0
0	I	6¾	4¼	0	0	0	0
0	V	6¼	5	0	0	0	0
0	V	9¼	3¾	0	0	0	0
2	I	4¾	6	4	4	0	4
2	I	6¾	4¼	3	0	0	8
2	V	6¼	5	7	4	2	4
2	V	9¼	3¾	3	10	0	3
4	I	4¾	6	9	Ruptured in 4 months	4	3
4	I	6¾	4¼	6	Ruptured in 19 months	3	1
4	V	6¼	5	5	Ruptured in 11 months	3	3
4	V	9¼	3¾	4	Ruptured in 8 months	1	15

such was the case. Especially noteworthy is the fact that all four failures occurred in specimens stored wet. Prisms stored wet might also be expected to be more erratic than those stored dry. Water initially available for corrosive action in specimens stored in a drying atmosphere would depend mostly on the amount used in the original mix, whereas the water available in the specimens stored wet would depend also on the ease with which water could percolate from the wet towels to the wire, a factor which would vary from specimen to specimen and from place to place in a given specimen. The results in Table 2 indicate that corrosion in specimens stored wet was more erratic than in those stored dry. Consequently the effects of certain variables are indicated better by the behavior of the dry specimens.

Stress-relieved wire was used in half of the prisms and as-drawn wire was used in the other prisms. Both wires were 0.162 in. in diameter and were purchased from the same manufacturer. The ultimate tensile strength of the stress-relieved wire was 258,000 psi and that of the as-drawn was 270,000 psi. The yield strength, 0.2 percent offset, was 211,000 psi for the stress-relieved wire and 213,000 psi for the as-drawn. All wires were tensioned initially to a load of 140,000 psi; the load was checked periodically and readjusted to 140,000 psi when necessary. After 3 months the load was reduced to 120,000 psi and maintained at this level for the remainder of the storage.

The dry specimens indicated that the stress-relieved wire corroded somewhat more than the as-drawn wire. The average reduction in tensile strength of stress-relieved wire embedded in concrete containing 2 and 4 percent calcium chloride was 5.1 percent; the reduction in strength of the corresponding as-drawn wire was only 1.6 percent. It

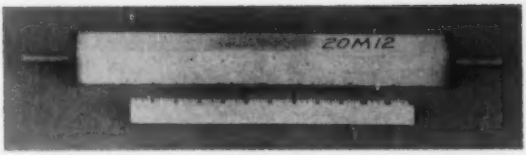

Fig. 9—Prestressed mortar specimen

should be noted also that all wire failures, which occurred in wet specimens, involved stress-relieved wire.

There was corrosion of the wire in all specimens which contained calcium chloride although in some cases the damage was not great enough to reduce the tensile strength significantly. There was no visible corrosion in specimens which were free of calcium chloride.

The average reduction in strength of wire embedded in concrete made with Type I cement plus calcium chloride and stored dry was 3.6 percent; the corresponding reduction for concrete made with Type V cement was 3.1 percent. This difference is not significant.

Two percent calcium chloride caused an average reduction in strength of the wire in dry specimens of 2.4 percent, and 4 percent calcium chloride caused a reduction of 4.4 percent.

Examination of the concrete which was adjacent to corroded wires showed that a severe spot of corrosion was always located adjacent to a large void in the concrete, i.e., a void with dimensions of the order of the wire diameter.

Corrosion in prestressed mortar specimens — The mortar and concrete in the two investigations just discussed were free to crack because the wire load was resisted, not by the mortar or concrete, but by external steel frames. To overcome this cracking tendency, 1¼ x 2 x 16-in. specimens, which were truly prestressed, were used in some additional studies. The wires were pretensioned by applying the loads through small metal plates which also served as the ends of the mortar mold. After the mortar had attained sufficient strength, the wire load was transferred to the mortar through these end plates. Fig. 9 shows one of these prestressed specimens. This particular specimen contained only one wire, but most specimens contained two wires which were spaced ⅝ in. center to center.

The mild steel end plates were electrically insulated from the wires to avoid extraneous electrolytic effects which previous measurements had shown are present when different metals are in contact with moist concrete. By way of a check on these previous findings, a few specimens with uninsulated end plates were stored for 1 year in a fog room. Even though some specimens contained 4 percent calcium chloride there was no corrosion of the wires. Apparently the potential between the mild steel and the wires was such as to corrode the mild steel and protect the wires.

Fig. 10—Corrosion products accumulated in a void which was adjacent to a wire

The wires used in the prestressed mortar tests were the same as those used in the 1 x 1 x 18-in. mortar specimens already described.

The mortar which consisted of 1 part cement, 2.5 parts sand, and from 0.42 to 0.45 parts water, required vibration for good placement. The first lift of the two lifts used in casting the specimens filled the mold up to the center line of the wires. A space was then formed under the wires by sliding a U-shaped piece of 1/16 in. wire along the under side of the prestressed wires. The first lift was allowed to stand for about 1 hr before the second lift was cast. This casting procedure assured the presence of large voids adjacent to the wires. Most specimens were steam cured for 16 hr at 140 F and were stored for 6 months with wet towels wrapped over the central portion.

Results are listed in Table 3, and are grouped according to the variables studied. Two of the specimens are repeated in each category to make comparisons easier. The first group was a study of the effect of using 0, 1, 2, and 4 percent calcium chloride. After 6 months storage under wet towels, the specimens were broken open and the wires and adjacent mortar surfaces were examined. The tensile strengths of the wires were then determined. The specimen which contained no calcium chloride showed no corrosion and the tensile strength was the same as in the original wire. Wires in mortar containing 1 percent calcium chloride did have spots of corrosion, but the reduction in tensile strength was not significant. The tensile strength of one of the wires embedded in mortar containing 2 percent calcium chloride was reduced 8 percent. The reduction in strength of the wires embedded in mortar containing 4 percent calcium chloride varied from 5 to 16 percent. Corrosion in mortars containing sodium chloride was about the same as that in mortars containing the equivalent amount of calcium chloride.

Although there were no outstanding differences in corrosion attributable to cement Types I, IS, III, or V, the greatest reduction in tensile strength, 32 percent, did occur in mortar made with Type V cement. There was no corrosion in composite specimens of Type I and V cements when no calcium chloride was used, and the corrosion was about the

TABLE 3 — CORROSION IN PRESTRESSED MORTAR SPECIMENS

Specimen No.	$CaCl_2$, percent	Type cement	No. of lifts	Specimen cross section, in.	Curing temperature, deg F	Storage	No. of wires	Kind of wire	Wire stress, 1000 psi	Reduction in strength, percent	
										Wire 1	Wire 2
Effect of calcium chloride											
1	0	I	2	1¼ x 2	140	6 mo. wet	2	A-SR	150	0	0
2	1	I	2	1¼ x 2	140	6 mo. wet	2	A-SR	150	0	0
3	2	I	2	1¼ x 2	140	6 mo. wet	2	A-SR	150	0	8
4	4	I	2	1¼ x 2	140	6 mo. wet	2	A-SR	150	8	9
5	4	I	2	1¼ x 2	140	6 mo. wet	2	A-SR	150	5	16
6	4 NaCl equiv.	I	2	1¼ x 2	140	6 mo. wet	2	A-SR	150	8	5
Effect of type of cement											
4	4	I	2	1¼ x 2	140	6 mo. wet	2	A-SR	150	8	9
5	4	I	2	1¼ x 2	140	6 mo. wet	2	A-SR	150	5	16
7	4	IS	2	1¼ x 2	140	6 mo. wet	2	A-SR	150	8	<7*
8	4	III	2	1¼ x 2	140	6 mo. wet	2	A-SR	150	0	11
9	4	V	2	1¼ x 2	140	6 mo. wet	2	A-SR	150	<5*	32
10	0	I&V	2	1¼ x 2	140	6 mo. wet	2	A-SR	150	0	0
11	4	I&V	2	1¼ x 2	140	6 mo. wet	2	A-SR	150	1	11
Effect of voids											
4	4	I	2	1¼ x 2	140	6 mo. wet	2	A-SR	150	8	9
5	4	I	2	1¼ x 2	140	6 mo. wet	2	A-SR	150	5	16
12	4	I	1	1¼ x 2	140	6 mo. wet	2	A-SR	150	<2*	<3*
13	4	I	1	1¼ x 2	140	6 mo. wet	2	A-SR	150	<2*	<1*
Effect of curing temperature											
4	4	I	2	1¼ x 2	140	6 mo. wet	2	A-SR	150	8	9
5	4	I	2	1¼ x 2	140	6 mo. wet	2	A-SR	150	5	16
14	4	I	2	1¼ x 2	73	6 mo. wet	2	A-SR	150	0	30
15	4	I	2	2 x 2	73	6 mo. wet	2	A-SR	150	1	13
Effect of storage											
16	4	I	2	1¼ x 2	140	1 day	2	A-SR	150	0	1
17	4	I	2	1¼ x 2	140	2 wk. wet	2	A-SR	150	0	2
4	4	I	2	1¼ x 2	140	6 mo. wet	2	A-SR	150	8	9
5	4	I	2	1¼ x 2	140	6 mo. wet	2	A-SR	150	5	16
18	4	I	2	1¼ x 2	140	6 mo. fog	2	A-SR	150	1	0
19	4	I	2	1¼ x 2	140	6 mo. dry	2	A-SR	150	1	1
Effect of number of wires											
4	4	I	2	1¼ x 2	140	6 mo. wet	2	A-SR	150	8	9
5	4	I	2	1¼ x 2	140	6 mo. wet	2	A-SR	150	5	16
20	4	I	2	1¼ x 2	140	6 mo. wet	1	A-SR	150	8	
Effect of kind of wire											
4	4	I	2	1¼ x 2	140	6 mo. wet	2	A-SR	150	8	9
5	4	I	2	1¼ x 2	140	6 mo. wet	2	A-SR	150	5	16
21	4	I	2	1¼ x 2	140	6 mo. wet	2	A-as-drawn	150	18	7
22	4	I	2	1¼ x 2	140	6 mo. wet	2	A-PCA	150	0	8
23	4	I	2	1¼ x 2	140	6 mo. wet	2	A-PCA	150	0	14
24	4	I	2	1¼ x 2	140	6 mo. wet	2	Regina	150	6	21
25	4	I	2	1¼ x 2	140	6 mo. wet	2	Regina-PCA	150	19	9
Effect of wire stress											
4	4	I	2	1¼ x 2	140	6 mo. wet	2	A-SR	150	8	9
5	4	I	2	1¼ x 2	140	6 mo. wet	2	A-SR	150	5	16
26	4	I	2	1¼ x 2	140	6 mo. wet	2	A-SR	15	<1*	<2*

*Failed outside mortar.

same as that with a single type cement when 4 percent calcium chloride was used.

The importance of voids adjacent to the wires was demonstrated by comparing the corrosion in two-lift specimens, which contained "built-in" voids, with the corrosion in comparable specimens cast in one lift. There was good contact between mortar and wire in the one-lift specimens. Both the wire strengths and visual examination showed that there was little corrosion in the specimens which had good wire to mortar contact. Throughout all of the studies, severe corrosion has been observed to be associated with large voids adjacent to the wire. Fig. 10 shows the accumulation of corrosion product in such a void.

The next group of specimens in Table 3 indicates that corrosion in specimens cured at room temperature may be just as severe as that in similar specimens cured in steam at 140 F. Also, with voids present, an increase in minimum cover from ¼ to ⅝ in. did not decrease the corrosion significantly. The minimum cover on the wires in specimens with a cross section of 1¼ x 2 in. was ¼ in., and that for the specimen with a cross section of 2 x 2 in. was ⅝ in.

Corrosion was plainly visible in specimens containing 4 percent calcium chloride after only 1 day although there was no significant reduction in strength. The insignificant reduction in strength of the wires in a specimen stored in a fog room for 6 months probably did not indicate that fog-room storage was any less severe than wet towel storage, but indicated rather that in a completely saturated specimen the mild steel end plates protected the wire electrolytically even though there was no direct metal to metal contact.

The purpose in using two wires in most tests was to determine if differences in composition or physical states between adjacent wires might lead to more severe corrosion than would occur with a single wire. The reduction in strength of a specimen containing a single wire was 8 percent or about the same order as the comparable two-wire specimens. However, there were several cases in which one wire of a specimen was severely corroded while the other wire was practically unaffected. Inter-wire potentials might be involved in such occurrences.

The next group in Table 3 lists results obtained on various types of prestressing wire, and indicates that the corrosion of stress-relieved wire, wire as drawn by the manufacturer, and wire drawn in our laboratory was not greatly different. Some wire from the Regina project exhibited perhaps slightly greater corrosion than the other wires. As noticed in previous studies, however, all the wires which were drawn in our laboratory unlubricated through an uncooled die, developed longitudinal cracks in the corrosive environment of concrete containing calcium chloride. Fig. 11 is a photograph of a Regina wire that was drawn in our laboratory from 0.162 to 0.142 in. and was embedded in mortar containing 4 percent calcium chloride for 6 months. The longitudinal

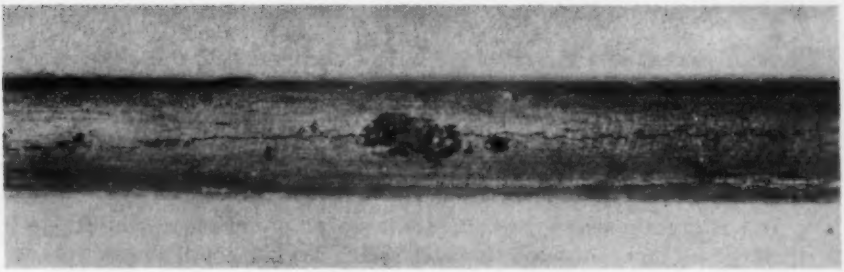

Fig. 11—Longitudinal crack in hard-drawn wire

crack in the wire apparently started at a spot of corrosion adjacent to a void in the mortar and then extended both ways along the wire.

The reduction in tensile strength of wires which were loaded to only 15,000 psi was considerably less than the reduction in wires which were loaded to 150,000 psi.

Although the results of Table 3 show some wide variations in corrosion of comparable specimens, the studies have been valuable in indicating in a general way the effects of several factors.

CONCLUSIONS

Field experience and laboratory studies have shown that the use of 3 to 4 percent calcium chloride or more in concrete containing prestressed wire may lead to serious corrosion or even rupture of the wire. Significant reductions in tensile strength of prestressed wires embedded in laboratory concrete specimens containing 2 percent calcium chloride were measured after a storage period of 2 years.

The possibility of the existence of a threshold amount of calcium chloride below which significant corrosion does not occur has been suggested. Our laboratory studies indicate that chloride ions probably persist in concrete for long periods of time even when the initial amount is less than 1 percent, and consequently corrosion would seem to be a possibility. The evaluation of the hazards of using small amounts of calcium chloride, say less than 2 percent, is difficult in short time laboratory tests. Such tests are valuable for showing that under certain conditions serious corrosion of prestressed wire may occur, but are not sufficient for demonstrating the safety of a given practice simply because of negative results. The demonstration of safe practice requires knowledge gained from extensive and long-time field experience.

In view of the present knowledge, the following conclusions seem to be justified:

1. Under normal conditions, conventional portland cement concrete provides excellent protection against corrosion of prestressing wire.

2. Corrosion of prestressing wire may occur in an environment of portland cement concrete when chloride ion is present.

3. The corrosion of prestressing wire in concrete containing chloride ions

(a) is an erratic process, and consequently quantitative evaluation is difficult

(b) is only slightly, if at all, dependent upon the type of cement used in the concrete

(c) is not greatly influenced by curing temperature

(d) is greater when the concrete is stored under moist conditions than when the concrete is stored under drying conditions

(e) may be appreciable, however, even when the concrete is stored under drying conditions

(f) is usually associated with large voids adjacent to the wire, i.e., voids with dimensions of the order of the diameter of the wire

(g) may be either increased or decreased by other metals in contact with the concrete

(h) may occur whether the wire is or is not subject to external load

(i) may occur whether the wire is hard-drawn or stress-relieved

(j) and may involve longitudinal cracking tendencies which are dependent upon the wire drawing conditions

4. Because of the hazard involved, it is recommended that calcium chloride not be used in prestressed concrete.

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Resistance to Shear of Reinforced Concrete Beams

Part 4 — Behavior of Beams with Different Types of Web Reinforcement

By A. M. NEVILLE and J. TAUB

A comparison of beams with different types of web reinforcement shows that bars inclined at 45 deg to the axis of the beam are most effective in resisting the diagonal tension stresses. To offer full protection from shear, however, the use of vertical stirrups and bent-up bars combined is preferable. The alternative of orthogonal web reinforcement, consisting of vertical stirrups and horizontal bars at, and possibly also below, the neutral axis, is discussed and experimental data are presented.

■ PART 2 OF THIS PAPER* discussed the behavior of beams reinforced by means of vertical stirrups only. Part 3 dealt with beams with bent-up bars or inclined stirrups. In this part it is proposed to consider first the behavior of beams in which both these types of web reinforcement are used. This will be followed by a discussion of other systems of web reinforcement.

Notation

a = shear span
d = effective depth of beam
f_c' = compressive strength of concrete test cylinder
f_{cu} = compressive strength of concrete test cube
jd = lever arm of internal forces in beam
kd = depth of compression zone of concrete
P_c = total load on beam at formation of first diagonal tension crack
P_u = total load on beam at failure
p = percentage tension steel area
r = percentage web reinforcement area
S = resultant stirrup force
T = force in main tension reinforcement
T_d = resultant of principal tension stresses below neutral axis
T_h = horizontal component of T
T_v = vertical component of T
V = total shearing force
V_c = shearing force resisted by concrete above neutral axis
v = nominal shearing stress in concrete
v_c = nominal shearing stress at formation of first diagonal tension crack
v_u = nominal shearing stress at failure

*Part 1 appeared in Aug. 1960 ACI JOURNAL, p. 193; Part 2, Sept. 1960, p. 315; Part 3, Oct. 1960, p. 443.

BEAMS WITH BENT-UP BARS AND VERTICAL STIRRUPS

The distribution of internal forces after the diagonal tension crack has opened in a beam whose web reinforcement consists of both bent-up bars and vertical stirrups is shown in Fig. 28. For equilibrium we have:

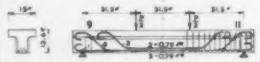
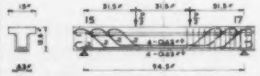
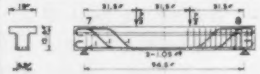
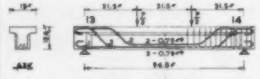
$$\left.\begin{aligned} T_v &= V - V_c - B_v - \Sigma S_v \\ \text{and} \qquad T_s &= C - B_s - \Sigma S_s \end{aligned}\right\} \quad \ldots\ldots (9)$$

It is apparent that the bent-up bars and the vertical stirrups help one another, as assumed by the various codes, but the effects of the two types of web reinforcement are more than additive.

A comparison of the behavior of T-beams with bent-up bars only and with bent-up bars and vertical stirrups was made by Saliger.[38]* Bach and Graf's data enable these authors to compare T-beams with stirrups only,[32] bent-up bars only,[18] and with both stirrups and bent-up bars.[18] All these beams were loaded at third points. Beams subjected to uniformly distributed loading were also tested without web reinforcement, with vertical stirrups, with bent-up bars, and with bent-up bars and stirrups.[22]

*References are numbered consecutively beginning with Part 1 of this five-part series.

TABLE 20 — COMPARISON OF SHEAR STRENGTHS OF BEAMS WITH BENT-UP BARS ONLY AND WITH BENT-UP BARS AND VERTICAL STIRRUPS

Investigator	Beam No.	BEAM DETAILS	a/d	f'_{cu} (c) psi	p percent	P_u kips	v_u psi	$\frac{v_u}{f'_{cu}}$	Increase in P_u due to provision of stirrups percent	Mode of failure
Saliger[38]	3		2.9	3560	3.6	37.4	306	0.086		Destruction of beam end by hooks
	5		2.9	3560	3.6	61.8	503	0.141	65	Balanced shear-tension and yield of steel at midspan
	9		2.9	3560	3.6	58.2	500	0.140		Destruction of beam end by hooks
	11		2.9	3560	3.6	65.2	560	0.157	12	Splitting of beam along the diagonal tension crack
	15		2.9	3560	3.6	67.0	573	0.160		Destruction of beam end by hooks
	17		2.9	3560	3.6	73.2	625	0.176	9	Balanced shear-tension and yield of steel at midspan
	7		2.9	3560	3.6	56.0	457	0.128		Destruction of beam end by hooks
	8		2.9	3560	3.6	65.2	533	0.150	16	Yield of steel at midspan
	13		2.9	3560	3.6	64.0	549	0.154		Destruction of beam end by hooks
	14		2.9	3560	3.6	68.0	582	0.163	6	Yield of steel at midspan
	19		2.9	3560	3.6	71.8	617	0.173		Yield of steel at midspan
	20		2.9	3560	3.6	74.0	635	0.178	3	Yield of steel at midspan

Notes: (c) Based on width of stem
All steel consists of plain round bars
Odd numbers refer to beams as shown in the left hand portion of the drawing; even numbers to the right hand side.

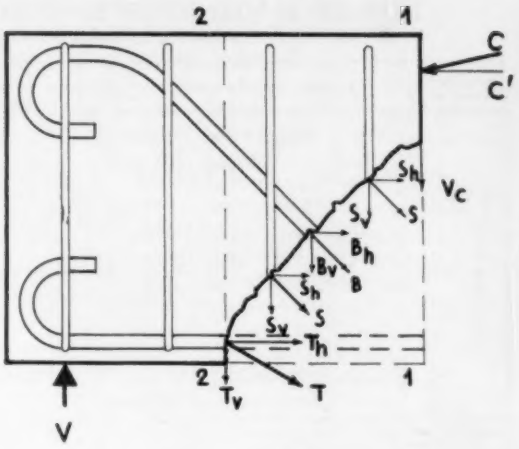


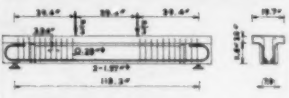
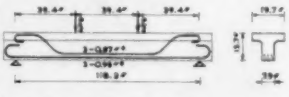
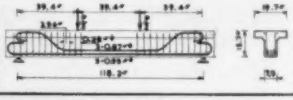
Fig. 28 — Distribution of internal forces after opening of the diagonal tension crack

The results of Saliger's[38] tests are summarized in Table 20. All the beams had the same cross-sectional area of tension steel at midspan but the size and number of bars were suited to the arrangement of bending up. From Table 20 it can be seen that the better the shear protection of the beam by bent-up bars the smaller the gain in strength from the addition of stirrups. This is in accordance with Eq. (9) stating that the forces in the bent-up bars and in the stirrups are additive. When the bent-up bars provide only a small degree of shear protection the role of stirrups is considerable.

The behavior of a beam with inadequate web reinforcement consisting of bent-up bars only has already been described in detail (see Part 2): the diagonal tension crack widens and results in the tension steel being pressed down; in the case of shear-tension failure the concrete at the level of the steel splits with a resulting loss of composite action in the outer parts of the beam. This leads to a premature failure of the beam as compared with its flexural strength.

When, however, vertical stirrups as well as bent-up bars are present the picture is entirely different: the stirrups carry a part of the diagonal tension and, therefore, relieve some of the force in the bent-up bars and in the tension reinforcement. The second function of the stirrups (cf. Part 2) then manifests itself: they prevent the pressing down of the tension steel at the lower end of the diagonal tension crack and the resulting splitting of the concrete, so that the composite action of the beam is preserved throughout its length. This enables the beam to resist a considerably higher load than when no stirrups are present, and the ultimate failure occurs either in shear-compression or, when the compression zone is sufficiently strong, flexure failure due to the yield of the tension steel takes place.

TABLE 21 — STRENGTH OF BEAMS WITH BENT-UP BARS AND VERTICAL STIRRUPS

Investigator	Beam No.	BEAM DETAILS	a/d	f'_{cu} (+) psi	P (++) percent	P_u kips	v_u psi	v_u/f'_{cu}	Mode of failure
Bach and Graf 32,18	16		2.9	3440	3.6	88.2	476	0.138	Destruction of beam end by hooks
	29		2.9	3440	3.6	92.6	500	0.145	Pressing down of main steel followed by destruction of beam end by hooks and crushing of concrete in the flange at midspan
	30		2.9	3440	3.6	107.0	577	0.166	Yield of steel at midspan

Notes: (+) Measured on 7.9 in. cubes
(++) Based on width of stem
All steel consists of plain round bars

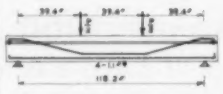
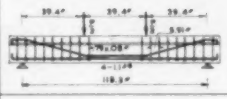
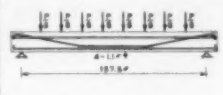
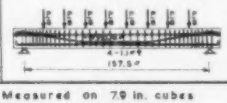
An experimental proof of this hypothesis concerning the role of stirrups is obtained by selecting some tests of Bach and Graf's,[32,18] summarized in Table 21, which shows also the crack patterns of the beams considered. Beam 30 has the bent-up bars of Beam 29 and the vertical stirrups of Beam 15 superimposed on one another.

In Beam 29, whose web reinforcement consisted of bent-up bars only, the opening of the diagonal tension crack was followed by the splitting of the concrete at the level of the tension steel. The diagonal tension crack opened wide because its width depends not only on the magnitude of slip movement in the part of the beam nearest to the support but also on the strain in the tension steel. This strain was constant over the length of the split part of the beam and, as a consequence, the full force in the tension bars was transmitted to the hooks.

In Beam 30, where vertical stirrups were provided in addition to the bent-up bars, a diagonal tension crack opened in the same place as in Beam 29 but the stirrups prevented the splitting of the concrete at the level of the tension steel, and the bond between the steel and the concrete was largely preserved. Owing to the action of the stirrups the force in the tension steel at the bottom of the diagonal tension crack was lower than in Beam 29, and decreased toward the support. Thus the strain in the tension steel was lower, too, and the diagonal tension crack remained fine. The composite action of the beam was preserved, and it took an increasing load until failure finally took place by the yielding of the tension steel in the zone of pure bending moment. Thus the addition of vertical stirrups prevented shear failure and, as can be seen from Table 21, increased the load carrying capacity of the beam by 16 percent.

There is one more interesting point about Beam 30: it shows how to arrange web reinforcement when, due to the distribution of the bending

TABLE 22 — COMPARISON OF STRENGTHS OF BEAMS WITH VERTICAL STIRRUPS AND BENT-UP BARS AT FLAT SLOPE AND WITH BENT-UP BARS ONLY

Investigator	Beam No.	BEAM DETAILS	f_{cu} (+) psi	p (++) percent	P_u kips	v_u psi	$\frac{v_u}{f_{cu}}$	Increase in P_u due to provision of stirrups percent	Mode of failure
Bach and Graf [18,22]	25		3440	3.6	76.1	411	0.119		Opening of hooks
	27		3440	3.6	99.0	534	0.155	30	Opening of hooks
	55		3440	3.6	73.6	397	0.115		Pressing down of main steel followed by opening of hooks
	56		3440	3.6	101.0	544	0.158	37	Yield at midspan

Notes: (+) Measured on 7.9 in. cubes
(++) Based on width of stem
All steel consists of plain round bars

moment over the length of the beam, it is necessary to bend-up the bars farther from the supports than would be desirable from a consideration of the shearing force; this is frequently the case in continuous and restrained beams. Although Beam 30 was simply supported, its arrangement of bent-up bars corresponded to such a case, and the stirrups supplemented the web reinforcement. As mentioned earlier, this combination of bent-up bars and vertical stirrups has resulted in a full protection from shear.

Similar conclusions regarding the use of the combined web reinforcement can be drawn from recent tests of Rüsch's.[39] He found that a rectangular beam with vertical stirrups spaced at ¼ jd approximately, and subjected to two quarter-point loads, giving the a/d ratio of 1.7, failed in shear. A T-beam with a web only 2/3 of the width of the rectangular beam but with twice the cross-sectional area of tension reinforcement ($p = 1.33$ percent) had bent-up bars in addition to the vertical stirrups. Despite a narrower web and also despite the increased compression and tension zones the T-beam failed in flexure owing to the presence of the combined web reinforcement, thus showing once again that bent-up bars and vertical stirrups combined give the most effective shear protection.

In their experiments on bent-up bars inclined at a small angle to the axis of the beam Bach and Graf[18,22] tested also some beams with vertical stirrups added to the bent-up bars. These beams showed an increase in the load-carrying capacity of between 30 and 37 percent over the beams

with bent-up bars only, as can be seen from Table 22. In addition to confirming the conclusions of the previous paragraph these tests emphasize the inadequacy of flat-slope bent-up bars. The behavior of beams with bent-up bars inclined at less than 45 deg is discussed in general later in the paper.

BEAMS WITH ORTHOGONAL WEB REINFORCEMENT

It is proposed now to compare the behavior of beams reinforced by means of vertical stirrups only with the behavior of beams with bent-up bars or inclined stirrups only. Such comparisons have been attempted in the past. For instance, Evans' tests[40] show bent-up bars to be more effective than vertical stirrups in maintaining the bond between the steel and concrete, and in minimizing the deflection of beams due to the opening of diagonal tension cracks. The inclined stirrups were a little less effective than bent-up bars but more so than vertical stirrups.

Elstner, Moody, Viest, and Hognestad's tests[3] on rectangular restrained beams with vertical stirrups and with inclined stirrups, at the same horizontal spacing, show that, taking average values, the inclined stirrups give an approximately 35 percent higher ultimate load than the vertical stirrups. Wilby[17] also found 45 deg inclined stirrups to be more effective than vertical stirrups, particularly with a higher percentage area of web reinforcement.

An interesting comparison is afforded by Bach and Graf's tests,[82,18] which extended over several decades. Beams 38, 42, and 47, described in Part 3 of this paper, were reinforced against shear by means of bent-up bars of a suitable cross-sectional area and placed at a correct spacing. These beams were fully protected against shear, and they failed in flexure-tension at midspan. On the other hand, all the beams with vertical stirrups only failed in shear long before the yield point of the tension steel was reached (see Table 9, Part 2). This happened even when the percentage area of web reinforcement, r, was as high as 1.8 percent (Beam b) although 1.25 percent would have sufficed according to the German code.

This shows then that *vertical stirrups alone* cannot always economically guarantee a full protection from shear failure because they do not

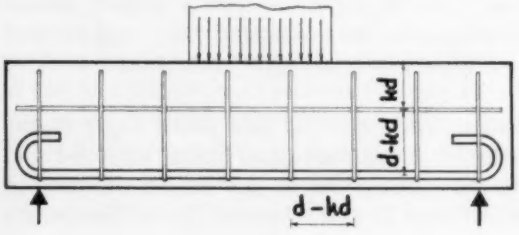


Fig. 29 — Orthogonal web reinforcement: one layer of horizontal reinforcement

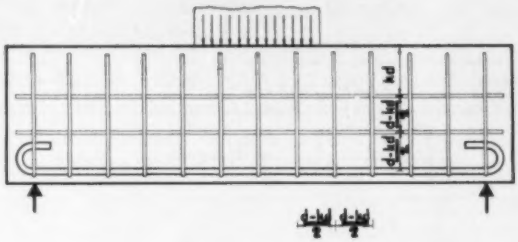


Fig. 30 — Orthogonal web reinforcement: two layers of horizontal reinforcement

lie in a direction normal to the diagonal tension crack. The realization of this fact has lead to the suggestion of a new system of web reinforcement wherein vertical stirrups are combined with horizontal bars capable of resisting the horizontal component of the principal tension. Because the resulting web reinforcement is placed in two directions at right angles to one another it is called orthogonal reinforcement.

Fig. 29 and 30 show two examples of orthogonal web reinforcement. In the former, one layer of horizontal reinforcement has been added at the level of the neutral axis, the vertical stirrups being spaced at $d\ (1 - k)$. In Fig. 30 two layers of horizontal reinforcement are present, one at the level of the neutral axis, the other half-way between the neutral axis and the main tension steel; the stirrups are spaced at $\frac{1}{2}\ d\ (1 - k)$. This layout would be used in a beam subjected to a high shear. It can be seen that in both cases a "square mesh" of web reinforcement is present.

To study the role of the orthogonal reinforcement Fig. 31 should be considered; it shows a free-body diagram similar to those used earlier. The principal tension T_d can be resolved into two components T_d' and T_d''. The former is taken by the stirrup a and by the longitudinal reinforcement, while the component T_d'' is taken by the stirrup b and by the additional (horizontal) shear reinforcement, which must, therefore, be no higher than a distance $d\,(1 - k)$ above the tension steel.

After the formation of the diagonal tension crack the stirrups a and b have to carry primarily only the vertical forces; the stress in the stirrups

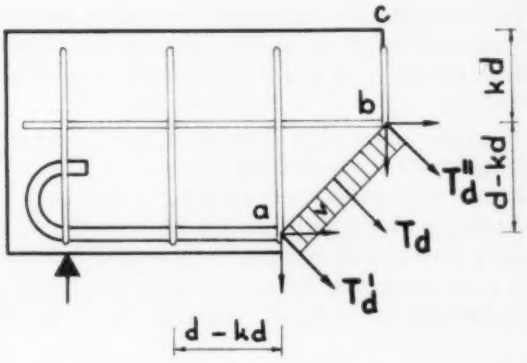
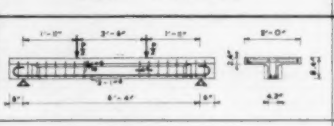
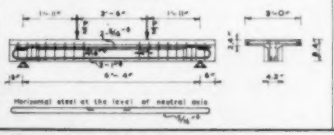


Fig. 31—Free-body diagram of a beam with orthogonal web reinforcement

TABLE 23 — STRENGTH OF BEAMS WITH ORTHOGONAL WEB REINFORCEMENT: SERIES I

Beam No.	BEAM DETAILS	$\frac{a}{d}$	f'_{cu} (a) psi	P_c kips	v_c psi	P_u kips	v_u psi	Ratio of total weight of reinforcement of beam with horizontal bars to beam without such bars
1		3.3	3050	15.8	307	31	602	
2		3.3	3050	21.8	424	39	758	1.06

Note: (a) Measured on 12 in. cubes
All steel consists of plain round bars

is, therefore, smaller than in a beam without the additional horizontal reinforcement. Likewise, the horizontal shear reinforcement reduces the tensile force in the main longitudinal reinforcement. It follows that the destructive action of the force in the main steel at the lower end of the diagonal tension crack in a beam with orthogonal web reinforcement is smaller than in a beam with vertical stirrups only. The load carrying capacity of the former beam must, therefore, be higher than that of the latter.

An experimental verification is afforded by three series of tests, performed in 1947, 1957, and 1958 respectively, and summarized in Tables 23, 24, and 25.

In the first series (Table 23) T-beams differing only by the presence of horizontal web reinforcement were compared; two companion specimens for each layout of reinforcement were tested, and the figures given in Table 23 are average values. The same table shows also the details of the reinforcement, and it may be noticed that the vertical stirrups and the horizontal reinforcement were of the same diameter; the latter was in the form of a large loop in the plane of the neutral axis. In the beams with vertical stirrups only the opening of the diagonal tension cracks was followed by the cracks extending into the flange; eventually some horizontal splitting at the level of the tension steel occurred, and the final failure took place due to the destruction of the beam end by the hooks.

In the beams with orthogonal web reinforcement the additional horizontal steel restricted the upward spread of the diagonal tension cracks so that it was only under a load of 37 kips that the crack penetrated into the flange. With increase in load both the diagonal tension crack and the flexure cracks at midspan widened, and failure occurred eventually due to a simultaneous failure in shear and yield of the tension steel in the center part of the beam. The addition of the horizontal steel has resulted

in a 25 percent increase in the load carrying capacity of the beam, the increase in the weight of steel used being barely 6 percent.

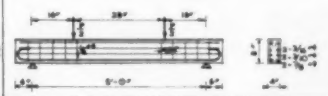
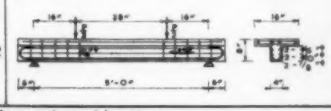
It may be interesting to note that the calculated value of the stress in the tension steel of the beams with orthogonal reinforcement exceeded the yield stress of that steel, so that flexure-tension failure was unavoidable.

The tests of the second series are summarized in Table 24, and the influence of the addition of a small amount of horizontal web reinforcement on the load carrying capacity of a beam can be clearly seen. This applies to both rectangular and to T-beams. In this series the horizontal reinforcement consisted of plain straight bars as distinct from the loops of Series I and III.

In Series III, T-beams with three different layouts of web reinforcement were compared, duplicate tests being made throughout; the values given in Table 25 are average values. Once again, the influence of the addition of one and two layers of horizontal web reinforcement on the failure load is clear. It may be interesting to note that in Beam 3a (Fig. 34) the flexure-tension crack below the left-hand load point widened considerably and extended through half the depth of the flange; this means that either a simultaneous failure in shear and flexure took place or tension failure was imminent when the collapse in shear took place. The crack patterns of Beams 1a and 2a are shown in Fig. 32 and 33, respectively.

It should be mentioned that there are some differences in the concrete strength of different beams in each series. From the data of Parts 1 and 2

TABLE 24 — STRENGTH OF BEAMS WITH ORTHOGONAL WEB REINFORCEMENT: SERIES II

Beam No.	BEAM DETAILS	$\frac{a}{d}$	f_{cu} (a) psi	P_c kips	v_c psi	P_u kips	v_u psi	Ratio of total weight of reinforcement of beam with horizontal bars to beam without such bars
R33		2.37	3600	11.2	235	19.7	413	
R42		2.37	4850	15.7	330	29.1	611	1.11
T33		2.37	5720	15.7	330	31.0	657	
T42		2.37	6350	16.8	353	38.1	808	1.09

(a) Measured on 3 in. cubes
All steel consists of plain round bars

Fig. 32 — Beam 1a (Table 25)

of this paper it is clear that the influence of concrete strength alone cannot account for the observed differences in failing loads. Furthermore, a comparison of Beams 1 and 3 of Series III shows that a beam with two layers of horizontal web reinforcement failed under a considerably higher load although it was made of weaker concrete than a beam with vertical stirrups only.

In 1953 Rausch[41] discussed the possibility of the use of orthogonal shear reinforcement on the basis of the multiple truss analogy. Walther[42] compared in 1956 beams with vertical stirrups and bent-up bars and beams with orthogonal web reinforcement, using three types of steel: polished round bars, threaded bars, and medium tension steel (Tor 40). In each beam the stirrups in one-half of the beam were of the same type of steel as the main reinforcement while in the other half of the beam the stirrups were always of mild round steel.

The beams with polished round bars (with hooks) failed in bond, the beam ends being destroyed by the hooks, but all the other beams failed in flexure-tension. This makes it difficult to draw any conclusions regarding the efficiency of orthogonal web reinforcement. Moreover, the comparison was made with beams with both bent-up bars and vertical stirrups. There is little doubt that this latter is an efficient type of web reinforcement but it is also expensive in labor costs and the provision of bent-up bars is often not possible over a sufficient length of the beam.

Fig. 33 — Beam 2a (Table 25)

Fig. 34 — Left-hand portion of Beam 3a (Table 25)

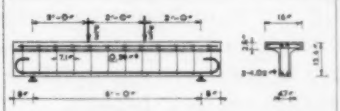
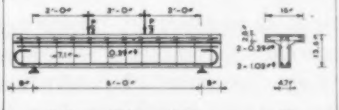
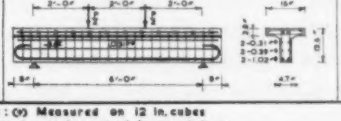
It is precisely because of this that the orthogonal web reinforcement is thought to offer a convenient solution.

No conclusion can be drawn from the actual values of ultimate load because the beams with bent-up bars and vertical stirrups had five 0.75-in. diameter bars as main reinforcement, while in beams with orthogonal web reinforcement only three such bars were present at the bottom of the beam, the other two bars (horizontal web reinforcement) lying one-third of the effective depth of the beam higher. Thus the moments of resistance of the two types of beams were not the same.

It may be recalled that the stirrups in Walther's beams were made of different types of steel in two halves of the beam; for instance, in a beam with threaded main steel the stirrups were of plain round steel and of threaded steel. It is interesting to note that the measured strain in the stirrups was the same for the two types of reinforcement.

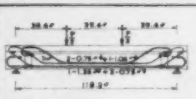
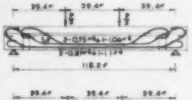
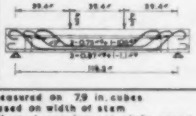
It should be noted that when there is more than one layer of web reinforcement present the longitudinal tension in the bars below the neutral axis, arising from the applied bending moment, has to be taken

TABLE 25 — STRENGTH OF BEAMS WITH ORTHOGONAL WEB REINFORCEMENT: SERIES III

Beam No.	Beam Details	a/d	$f_{cu}^{(a)}$ psi	P_c kips	v_c psi	P_u kips	v_u psi	Ratio of total weight of reinforcement of beam with horizontal bars to beam without such bars
1		2	3080	32.6	330	57.3	580	
2		2	3870	29.2	296	64.0	648	1.09
3		2	2770	32.5	329	73.5	744	1.21

Note: (a) Measured on 12 in. cubes
All steel consists of plain round bars

TABLE 26 — SHEAR STRENGTH OF BEAMS WITH BENT-UP BARS AT 30 AND 45 DEG

Investigator	Beam No.	BEAM DETAILS	a/d	f'_{cu} (cc) psi	p (cc) (+) percent	P_u kips	v_u psi	$\frac{v_u}{f_{cu}}$	% of beams with 30° bent-up bars as a percentage of beams with 45° bars	Mode of failure
Bach and Graf	43		2.9	3440	3.6	90.0	485	0.141	89	Splitting of concrete by hooks of bent-up bars
	36		2.9	3440	3.6	101.5	547	0.159	100	Splitting of concrete by hooks of straight bars, followed by those of bent-up bars
	44		2.9	3440	3.6	100.0	540	0.157	91	Splitting of concrete by hooks of bent-up bars
	38		2.9	3440	3.6	109.2	590	0.172	100	Yield of steel at mid-span

Notes: (cc) Measured on 7.9 in. cubes
(+) Based on width of stem
All steel consists of plain round bars

into account. The use of two or more layers of horizontal web reinforcement (with a corresponding spacing of vertical stirrups) is advisable when high shearing forces may act upon the beam. Orthogonal reinforcement gives a uniform protection against shear over the full length of the beam; furthermore, the designer is not confronted with the difficulty of providing the correct amount of tension steel and a sufficient number of bent-up bars at the same time. Orthogonal reinforcement in more than one layer is also particularly suitable when a heavy concentrated load acts near a support or where a short beam is subjected to a high shearing force but to a comparatively low bending moment. The alternative of vertical stirrups and bent-up bars inclined at an angle greater than 45 deg to the axis of the beam is less economical and less effective.

The uniform protection against shear throughout the length of the beam, provided by the orthogonal web reinforcement, is of considerable importance because, as shown in Parts 2 and 3, in beams subjected to uniform or unsymmetrical loading and reinforced against shear in the zone of a high shearing force only, failure in shear can occur where the shearing force acting is lower than the maximum on a beam.

The use of orthogonal web reinforcement in continuous beams would also appear to be particularly advantageous: the difficulties of using bent-up bars in the part of the beam where the point of contraflexure may occur are avoided, and the tension reinforcement can be placed entirely to suit the bending moment. Furthermore, the cold-drawn high tensile steel, increasingly used as concrete reinforcement, is difficult to bend and, therefore, unsuitable for the provision of bent-up bars. Web reinforcement consisting of vertical stirrups alone is not sufficient and,

in fact, not permitted by the ACI Code when the nominal shearing stress exceeds 0.08 f_c' or 240 psi.

BEAMS WITH BENT-UP BARS INCLINED AT LESS THAN 45 DEG TO HORIZONTAL

In shallow beams it is sometimes convenient, or even necessary, to bend up bars at an angle considerably less than 45 deg to the axis of the beam. The use of such web reinforcement can be evaluated by a comparison with standard 45 deg bent-up bars, described in Part 3 of this paper.

Table 26 gives the data for some of Bach and Graf's[18] beams with bent-up bars at 30 and 45 deg to the axis of the beam. All the reinforcement layouts are based on the double lattice system. The higher load carrying capacity of beams with the 45-deg bars is apparent. When the web reinforcement of a beam consists solely of bent-up bars inclined to the axis of the beam at less than 45 deg the principal tension cannot be wholly resisted by the bent-up bars: there is a vertical component of this tension which must be taken by the concrete and the main steel.

The principal tension can be resisted wholly by the web reinforcement if, in addition to flat-slope bent-up bars, vertical stirrups are provided. This has been proved experimentally (Table 22) and was discussed earlier in this paper, but the inadequacy of flat-slope bent-up bars must not be forgotten.

BEAMS WITH BARS INCLINED AT MORE THAN 45 DEG TO THE HORIZONTAL

By an argument similar to that applied to bent-up bars inclined at less than 45 deg to the axis of the beam, beams with bent-up bars at an angle greater than 45 deg to the horizontal are not satisfactorily reinforced against shear, since the bent-up bars cannot resist the whole of the principal tension. A part of the horizontal component of it is carried by the tension steel but if diagonal cracking is to be checked horizontal steel members are needed between the level of the main steel and the neutral axis. Beams with such orthogonal web reinforcement have been described earlier in the paper.

Beams with welded stirrups inclined at 45 and 67½ deg to the axis of the beam were tested by Moretto,[31] who concluded that the steeper stirrups were more effective as shear reinforcement. This statement, however, has to be considered with caution for in Moretto's tests the stirrups inclined at 67½ deg were spaced at 0.765 of the spacing of the 45-deg stirrups (measured along the beam axis) so that the number of the former was correspondingly greater. Furthermore, the concrete strengths varied from beam to beam thus vitiating direct comparison.

Some tests on stirrups inclined at 67½ deg were also made by Evans[26] but further tests are needed to assess the effectiveness of bent-up bars

inclined at different angles. These may be desirable in special cases but it is believed that, as a general rule, bars inclined at 45 deg to the axis of the beam are most effective.

COMPARISON OF INCLINED STIRRUPS AND BENT-UP BARS

No direct comparison of these two types of web reinforcement has been made, or is indeed possible, since the layout of the main reinforcement in the two types of beams cannot perforce be the same: the effects of cut off of the tension reinforcement in the tension zone were discussed in Parts 2 and 3.

It is possible, however, to compare the effectiveness of bent-up bars and inclined stirrups, as was done, for instance, by Evans.[40] For deep beams (span: depth ratio of 5) he found that bent-up bars preserved the composite action of the beam up to a very high load. On the other hand, inclined stirrups, and likewise vertical stirrups, led to an appreciable slip between the main steel and the concrete.

In beams of normal depth (span: depth ratio of 10) the bond was less affected in all types of beams, but here, too, the bent-up bars maintained the bond at higher loads than the inclined or vertical stirrups. Evans[40] concluded, therefore, that web reinforcement in the form of bent-up bars is the most effective type of reinforcement in both maintaining bond, and in minimizing the deflections of beams due to the opening of diagonal tension cracks. The subject of bond will be discussed more fully in Part 5.

It is not proposed to discuss the economic aspects of the comparison of bent-up bars and inclined stirrups as so much depends on the relative costs of material and labor. Ignoring, therefore, the cost of the material it may be observed that the use of inclined stirrups permits the main steel to continue up to the support, and this has been shown to increase the load-carrying capacity of a beam in shear.

It is important that inclined stirrups are well anchored. The importance of the anchorage at the top end was shown by tests described in Part 3 of this paper: the more securely the bent-up bars are anchored the higher the ultimate load of the beam. This may account, at least partially, for Evans' results.[40]

Inclined stirrups are less securely anchored at their lower ends than bent-up bars and are, therefore, less effective in taking over the horizontal force transmitted by the main steel; a slight horizontal movement between the stirrup and the tension steel is possible.

Similarly, when a diagonal tension crack opens and reaches the level of the tension steel the bent-up bars can partially resist the pressing down of this steel, while it may be thought that inclined stirrups, being able to accommodate a slight horizontal movement at their lower ends, are very much less effective in preventing the pressing down of the main steel.

CONCLUSIONS

An absolute comparison of beams with different types of web reinforcement is difficult, and sometimes impossible. It is possible, however, to infer from test results and a study of free-body diagrams that web reinforcement inclined at 45 deg to the axis of the beam is most effective in resisting the widening of the diagonal tension crack.

As an alternative, web reinforcement arranged in an orthogonal pattern of vertical stirrups and horizontal bars at and below the neutral axis can also resist all the forces in the direction of the principal stress. The authors' tests on beams with such reinforcement have confirmed its effectiveness.

Bent-up bars have been found to be more effective than inclined stirrups in preserving the composite action of the beam, primarily because of their better anchorage both at the level of the main steel and in the compression zone. If the bent-up bars are continued horizontally in the compression zone before terminating in a hook the resulting load carrying capacity of the beam is further increased.

There is, however, one property of vertical stirrups in which they are superior to bent-up bars: vertical stirrups can effectively prevent the pressing down of the tension steel at the bottom of the diagonal tension crack. For this reason the use of vertical stirrups and bent-up bars combined offers the most effective form of shear protection, and is strongly recommended. The orthogonal system of web reinforcement is also considered to be effective and is particularly useful in continuous and restrained beams where the use of bent-up bars only is not possible, and the use of vertical stirrups alone might not always prevent shear failure, although they would give a high ultimate strength of the beam.

Finally, it may be relevant to repeat that beams differing in web reinforcement only exhibit the first diagonal tension crack under the same load. It is only after the cracking has started that the behavior of a beam depends on the type of web reinforcement used, and the further development of cracks and the ultimate load are a function of this reinforcement.

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Monolithic Cast-In-Place Concrete Pipe

By L. H. KRISTOF

The development and use of monolithic cast-in-place concrete pipe is discussed, including the merits of this type of water carrying conduit. The various factors that should be considered in evaluating the practicability of the use of this type of conduit are outlined. General summary is made of advantages and limitations of different construction methods.

■ MONOLITHIC CAST-IN-PLACE CONCRETE PIPE discussed in this paper is a conduit constructed by placing and forming concrete directly in a trench with vertical sides and semicircular bottom. The pipe is constructed in an integrally complete or monolithic, jointless and substantially seamless concrete cylinder embedded directly in the bottom of the trench. The cross-sectional shape of the conduit is generally circular or nearly circular.

Briefly, the history of the development of this type of concrete pipe stemmed from the need for a low cost and low head underground water carrier conduit for irrigation and drainage uses. This conduit was originally developed during the late 1920's in the San Joaquin Valley of California — principally in the Turlock and Modesto Irrigation Districts in Stanislaus County. It was developed to eliminate numerous farm service irrigation and drainage ditches and canals; thereby reclaiming valuable agricultural land, together with various other benefits such as less requirement for right-of-way, easements, and reduction of operation and maintenance costs.

In recent years this conduit has also been used by a few municipalities for storm drains, generally outside of traffic lanes. Reinforcing steel should be used in the pipe wall when utilizing this conduit under roadways, streets, and other critical locations. There is a growing tendency to use unreinforced cast-in-place concrete pipe in situations where risks from failure are high. The limitations of cast-in-place concrete pipe should be carefully considered before selecting it for use.

DESCRIPTION

Monolithic cast-in-place concrete pipe is not a conventional concrete pipe laying process. It is not considered to be in the same category or classification as precast concrete pipe, regardless of any of the monolithic

concrete pipe construction methods developed to date, and is not considered a substitute for any of the economic uses of precast pipe. Also, it is not considered to compete with high head precast concrete conduits and the general broad range of uses for which the latter has been designed and developed.

Reinforcement

Unreinforced monolithic cast-in-place concrete pipe is a low-head conduit generally considered to be adequate for operating water pressure heads of 3 to 5 ft. Properly constructed pipe, under ideal conditions, is generally considered safe for pressure heads up to 8 ft. For use in excess of 8 ft and possibly up to a maximum of 12 ft pressure head, reinforcement steel should be utilized in the pipe shell to assure safety of the conduit. The use of this conduit for pressure heads above 12 ft is considered inadvisable without reinforcing steel because of variations in wall thickness and consolidation.

Due to limitations of vibrations during placement, the density of the pipe wall is often considerably less than in precast concrete pipe. The use of higher water pressure (above 12-ft head), can be expected to contribute to greater water seepage through low density or porous concrete pipe walls.

The principal reasons for use of reinforcing steel in this conduit are: (a) to enable greater depths of earth fill to be placed safely over the conduit; (b) for protection against possible water-hammer, excessive shock, and vibrations; (c) for protection against excessive external live loads and possible eccentric earth pressures against the conduit.

Common dimensions

The most commonly used pipe sizes are diameters of 24, 30, and 36 in. Pipe sizes of 42, 48, 54, and 60 in. have also been constructed but to a minor extent. The smallest pipe that is practical for construction is 24-in. diameter, since workmen are required to work inside the conduit. Diameters may vary as much as 5 percent and wall thickness may increase or decrease as much as ½ in. Requirements for pipe wall thickness vary. However, wall thickness used for the various pipe diameters is generally as follows:

Inside diameter, in.	Wall thickness, in.
24	2½ to 3
30	2½ to 3
36	3 to 3½
42	4 to 4½
48	5 to 5½
54	5½ to 6
60	6 to 6½

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Engineering experience has been in connection with design, construction, and operation of irrigation, drainage, and flood control. Employment in federal service has offered considerable experience in research, development, and evaluation of various water carrier systems.

Concrete

The different methods of constructing monolithic cast-in-place concrete pipe described in this paper are considered as essentially different processes of handling concrete to produce the same end product—all of which have about the same general limiting features, characteristics, or end results, whether constructed by hand or mechanical methods.

The typical concrete mix used in the various methods of pipe construction consists of approximately half sand and half aggregates in proportion by weight, with 5 to 6 sacks of portland cement per cu yd. The largest size aggregate generally used is 1 in., however, ¾ in. maximum is most commonly used. The use of larger size aggregate tends to make workability more difficult with hand construction methods, and tends to segregate more with machine methods.

The concrete slump most successfully used is in the range of 1½ to 2½ in. Consistency requirements are more critical for making the bottom half of the pipe with the two-stage pipe construction method, since the concrete must have a fairly low slump consistency such that the material will retain the semicircular shape of the pipe bottom. Concrete of this lower slump is also required in constructing the upper portion of the pipe with the various machine methods to reduce tendency of the concrete to slide off the oiled metal forms due to vibration of equipment (see Fig. 10). In using machine methods for pipe construction, especially with a coarse or harsh concrete mix, any tendency for aggregates to segregate is more critical than a limited variation in slump.

DISCUSSION

Some of the principal desirable or favorable features of monolithic cast-in-place concrete pipe are:

1. Low cost. In December, 1958, prices as low as the following were bid in the San Joaquin Valley of California for unreinforced monolithic concrete pipe:

30-in. diameter	$1.75 per linear ft
36-in. diameter	$2.00 per linear ft
42-in. diameter	$3.00 per linear ft

The above prices include excavation, furnishing all materials, constructing pipe, and placement of backfill. Also included is guaranteed satisfactory operation for 1 year.

Two-Stage Construction By He

Fig. 1 (left) — Trenching machine excavating trench with vertical walls and semicircular bottom for all methods of monolithic cast-in-place concrete pipe. The trench width and diameter of bottom conforms to the outside dimension of the lower half of the conduit. A backhoe with semicircular bucket attachment and other types of equipment are also used for excavating this type of trench. It is essential to avoid overhang in nonvertical trench walls to reduce bank caving during pipe construction. The surface of the trench bottom should be trimmed and shaped as neatly as possible to reduce excessive use of concrete

Fig. 2 — Concrete is being deposited on receiving platform of forming boat or trough with semicircular bottom for constructing lower half of concrete pipe. Workman on platform distributes the concrete around the sides and ahead of the boat and simultaneously rocks the equipment sideways as the workman in the trench bottom pulls the semicircular forming boat forward, thereby placing and shaping the bottom half of the concrete pipe. Semicircular metal forms for constructing the top half of the concrete pipe are being positioned in place in the immediate background. In far background completed pipe is covered with about 6-in. layer of moist soil as curing aid for concrete

Fig. 3 — Completed bottom half of the concrete pipe is shown in the foreground. A board is placed on the fresh concrete in the pipe bottom, which acts as a base for the wood vertical form supports. A workman also walks on this board when placing the metal semicircular forms in position. These metal forms are made of aluminum, and have been treated with a thin application of oil or grease on the outside surface. Construction of the top half of the pipe is followed immediately after a few feet of bottom is completed. Workman standing on metal forms in background spreads concrete with shovel and wood float for constructing top half of the pipe

By Hand Method

Fig. 4 (right)—Concrete is placed on metal forms for constructing top half. As concrete flows from the wheelbarrow, workman standing on metal forms distributes the material around the sides and top of the pipe. Final finish work is accomplished by hand troweling the pipe surface with a wood trowel. Completed length of concrete pipe is in foreground. The next step consists of covering the surface of the pipe with a 4- to 6-in. layer of moist soil for curing.

Metal semicircular forms along trench bank in background are spaced for use as required as pipe construction progresses.

Note four-man crew and foreman performing the entire pipe construction operation

Fig. 5 — View showing steel reinforcing cage placed in position for lower half of pipe. Concrete is being placed and formed with semicircular forming trowel or boat. Workman standing on the troweling form imparts lateral motion to the boat as it is simultaneously pulled longitudinally in the trench. The steel cage is positioned approximately in the center of the pipe shell. The reinforcing steel consists of 5/8-in. round bars

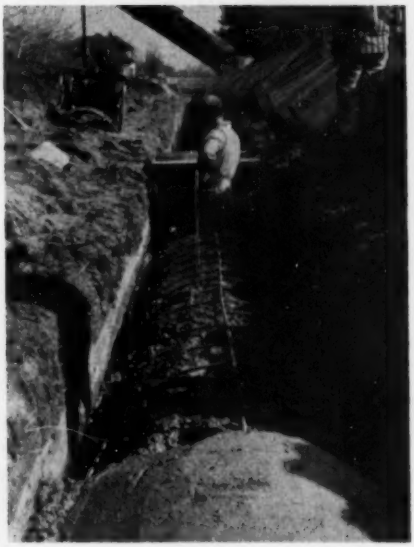

Fig. 6 — View showing the steel reinforcing cage placed in position for top half of pipe. Concrete for approximately one-half of the pipe shell thickness has been placed. The remaining concrete encasing the steel is hand placed, and hand troweled for final finishing operation. The reinforcing consists of 5/8-in. round bars spaced at 12-in. intervals. The semicircular bars embedded in the concrete of the pipe wall in the trench sides overlays the steel in the lower half of the pipe by about 10-in.

2. Minimum trench width is utilized. The required trench excavation conforms to the lower outside periphery of the conduit.

3. The direct contact or embedment of the bottom portion of the completed concrete conduit with the trend foundation is considered to be a highly desirable feature. Recent tests that have been conducted on the bedding of this type of conduit appear to indicate favorable results. In cases where the pipe is constructed in exceptionally firm trench foundations, it would appear reasonable to assume that the effects of this restraint and embedment would afford superior support and also minimize leakage from circumferential cracking.

4. Special considerations or techniques usually are not required for backfilling trench after pipe is completed. This conduit is principally used for agricultural purposes with average backfill depths of 18 to 36 in. in which special compaction or other refinements either have not been necessary, or customarily are not specified, to minimize costs. The maximum safe backfill depth over unreinforced pipe that customarily is used for average conditions is about 5 ft.

Some other salient aspects that should be considered in regard to monolithic cast-in-place concrete pipe are:

1. Trench conditions must be definitely favorable for this type of pipe construction. The soil in the trench walls must be firm and not subject to sloughing or caving. Vibration of pipe making and other equipment increases vulnerability to sloughing and caving of trench walls.

2. Densely compacted clayey and loamy sands, approaching hard-pan in cementation, form ideal trench foundations. Most sandy, gravelly, and loose soils are not adaptable for this type of construction. Thorough investigation should be made of soils and foundations to determine whether trench conditions will be as required for acceptable monolithic concrete pipe construction.

3. Ground water in the trench during pipe construction renders this method of conduit installation most difficult, if not impossible without well points. However, installation and maintenance of well points, for lowering ground water level to enable concrete to set sufficiently, may be prohibitive in cost. If adequate provision is not made for lowering ground water level in trench bottom, mud and water is apt to infiltrate into and contaminate the freshly placed concrete.

4. Where blasting of rock is required for trench excavation, the practicality of using this type of pipe construction is highly questionable. Blasting for excavation would be likely to cause excessive overbreak of trench sides and bottom, thereby requiring compacted refill. This refill work, in addition to probably being excessively expensive

due to additional labor and equipment use, may not be otherwise practical since vibration of pipe making equipment in the trench would be apt to loosen the refill soil, making pipe construction impossible. Requirements of trench shoring or bracing, if practical to use, during pipe construction would also be an added cost feature that would seriously affect the economy of this method of conduit construction.

5. Depending on various factors and considerations, the construction of relatively short installations of this pipe are not economical because of the cost of mobilizing and demobilizing.

6. There is no known record or experience available as to performance of monolithic cast-in-place concrete pipe under variable freezing and thawing ground conditions in different types of soils. Until such data are available, any installations should be considered as experimental.

7. The problem of soil particles becoming intermixed with concrete in the pipe wall during construction is an ever existing one. This contributes to structural weakness in the pipe shell and to seepage problems.

8. Since the conduit is constructed directly in place in the trench the concrete must be adequately cured and have sufficient strength before placement of backfill or any loads over the pipe can be accommodated, and before internal water pressure can be applied. This requires maintaining open trenches for a longer period of time.

9. This pipe has been primarily utilized under intermittent low pressure water flow conditions. Little data are available on the performance of this pipe under continuous full flow and pressure operation over an extended period of time. This pipe has been principally used over the years for irrigation and drainage purposes for farm service laterals, which require only intermittent use. Most of the land is flooded during irrigation operations for relatively short periods of time. Accordingly any leaks that may exist or develop in this conduit, used in this manner, are considered inconsequential.

10. It is not the intent or purpose of this paper to discuss the performance of monolithic cast-in-place concrete pipe under the various conditions where it has been used. Considerable information on this subject has been issued from various sources, much of which consists mainly of claims and data which is incomplete, confusing, and in some cases irrelevant. Much of this data tends to be partial and is subject to interpretation and evaluation. A paper on the performance of monolithic cast-in-place concrete pipe is currently being prepared. Briefly, cast-in-place concrete pipe has a definite and substantially established category of use. Beyond this there is a limit of practicability and safety, which should be mandatory for consideration before selecting this type of conduit for use.

CONSTRUCTION METHODS

Three methods of constructing monolithic cast-in-place concrete pipe are outlined herein and are referred to as follows:

1. Two-stage construction by hand method.
2. Two-stage construction by machine method.
3. One-stage construction by machine method. (There is currently being developed a new one-stage machine method of constructing cast-in-place concrete pipe by utilizing an inflatable rubber and fabric inner form. No data are available on this pipe construction method at this time. However, experimental and test work is being conducted.)

The rate of constructing monolithic concrete pipe by the various methods are generally about the same. Assuming all other factors to be equal, the major element influencing production is the rate of supply of concrete. Construction of 100 ft of 24-, 30- and 36-in. diameter pipe per hour is not uncommon.

The cost of monolithic concrete pipe constructed with the various methods vary considerably, depending on factors involved. However, to date the hand method of pipe construction has predominately been the most economical.

Two principal features are common to all three of the above methods of constructing monolithic cast-in-place concrete pipe. They are: (a) requirement of trench with vertical walls and semicircular bottom (Fig. 1); and (b) semicircular metal forms are required for forming the top portion of the pipe (Fig. 3, 9, 10, and 11). General procedures showing construction of this pipe by the three methods are illustrated and described in Fig. 1 to 14.

Two-stage construction by hand method

The original hand method of constructing monolithic cast-in-place concrete pipe in two stages is extensively utilized and is the major method employed to date. The two-stage method consists of constructing the conduit in two parts or stages. The lower half of the pipe is first constructed directly against the trench bottom, followed by placement of the top half. Various minor techniques with some of the construction operations and procedures vary among contractors. The procedure and general details of the hand method of constructing monolithic cast-in-place concrete pipe are illustrated in Fig. 2 to 6. It is estimated that over 200 miles of this pipe were constructed by this method during 1958 in the San Joaquin Valley of California alone.

Some of the pertinent reasons for popularity of the two-stage hand method of constructing monolithic cast-in-place concrete pipe are as follows:

1. No special pipe making equipment or patented process are required.
2. Ordinary labor is adequate.

3. All construction operations are performed with conventional construction equipment which can be readily modified when required.

4. Reinforcing steel if needed can easily be installed in the pipe wall during construction (Fig. 5 and 6).

5. Probability of trench caving and earth sloughing is reduced to a minimum during pipe construction since no vibrating equipment is used in the trench.

6. The procedures of construction are relatively simple, enabling numerous contractors to engage and compete in this type of work, thereby enhancing attainment of low prices.

Some of the possible limiting features of this pipe construction method that should be noted are as follows:

1. There is a tendency for loose soil from the trench bank and wall to fall on the two longitudinal side seams following construction of the lower half of the pipe. (Note the longitudinal seam of lower portion of pipe in Fig. 3.) A variation in technique utilizes a tapered edge for the longitudinal seam so that it will tend to shed any soil that falls from the sides and is shown in Fig. 10.

2. There is a tendency for the wood spreader supports or footings placed in the bottom of the pipe during construction to sink or become embedded in the concrete. (Note the wood spreader or footing in position in pipe bottom, supporting vertical brace, in Fig. 3.)

3. Different semicircular forms, for constructing upper half of pipe, generally are required for different pipe sizes. (However some contractors make dual use of upper metal forms such as shown in Fig. 10 where 30-in. diameter concrete pipe is being constructed with semicircular forms that can also be used to construct 36-in. diameter pipe.)

Two-stage construction by machine method

In this method of construction, a machine is used for casting the lower half of the pipe directly against the trench bottom, as in the hand method. A separate machine is used for placing the top portion of the conduit. The machines used by contractors for casting the bottom and top portions of the conduit vary in the principle of utilizing slip-form or wheel-guided type equipment or a combination of both. Fig. 7 to 11 illustrate the construction of two-stage monolithic cast-in-place concrete pipe by machine methods.

Some of the desirable features of this method of constructing monolithic cast-in-place concrete pipe are:

1. The machines used for construction are relatively simple, inexpensive, small, light in weight, and easy to handle.

2. Separate machines are used for constructing the top and bottom portions of the pipe. This reduces requirements of consolidating con-

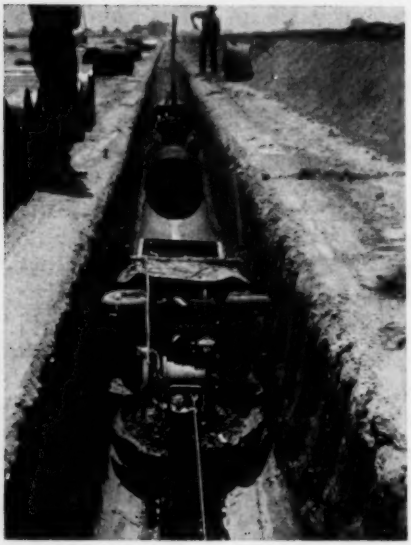

Fig. 7 — Machine in foreground constructs first stage or bottom half of pipe. Machine in background constructs second stage or top half of the pipe.

Machine for constructing bottom half of pipe is a traveling form actuated by a gasoline engine. Travel is accomplished by means of a drum and cable mechanism mounted on the machine. The cable is extended forward in the trench and is anchored. The front end of the machine is attached to a vertical shaft over a wheel which travels along the trench bottom ahead of the machine. This suspension mechanism is adjustable and enables various thicknesses of pipe wall to be made. The back end of the machine rests on an adjustable sled or runners which slide along the trench bottom. As concrete is placed in the hopper, the material flows around the periphery of the trench bottom without vibration. Final finishing is accomplished by means of a semicircular forming boat attached to the rear of the machine. A workman stands in this boat and applies a rocking motion to it as the equipment moves forward. The completed bottom of the concrete pipe is shown in the immediate background

Two-Stage Construction By Machine Method

Fig. 8 — Construction of top half of pipe is followed immediately after bottom portion is completed. Machine for constructing top half of pipe is placed in position over the oiled semicircular inside metal forms. This machine is likewise a traveling form which places and molds concrete for the top of the pipe. It is pulled forward by a gasoline engine and drum and cable mechanism. The cable is generally anchored to the forward traveling form. This machine is supported by rubber tire wheels in front and steel runners in the rear. Thickness of the pipe wall can be varied by adjustment of the wheels and runners. A steel tamper is provided for each side of the machine for tamping concrete as required. As concrete is placed in the hopper, the material flows around the periphery of the metal forms without vibration. Final finishing to the outside surface of the pipe is imparted by pulling a weighted rubber or canvas mat behind the machine. Note completed pipe in background

Fig. 9 — View showing a different slip-form machine for constructing bottom half of the pipe. The machine is a traveling form activated by a gasoline engine. Travel is accomplished by means of a drum and cable mechanism mounted inside the machine. The cable is extended forward in the trench and is anchored. The bottom of the machine is semicircular and conforms to the diameter of the trench bottom. As concrete is deposited in the hopper, the material is partitioned equally for each side of the trench and is placed directly on the trench bottom without vibration. Note the scoop in front of the machine which scrapes any loose soil on the trench bottom as the machine is pulled forward, thereby assuring firm contact of concrete pipe bottom with pipe foundation. This machine constructs tapered longitudinal seams for the edges of the pipe section as shown in Fig. 10. Concrete for top portion of pipe is being placed in background by hand method

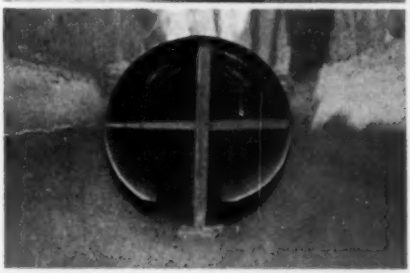

Fig. 10 — This pipe bottom was constructed by the machine shown in Fig. 9. Note the smooth surface imparted to the concrete. The tapered longitudinal seams or edges on each side of the concrete pipe section serve to deflect any soil that falls on these surfaces. These tapered seams also serve as an improved means for meshing the concrete of the lower half of the pipe with the top half. Construction of the top half of the pipe is followed immediately after a few feet of bottom is completed. Top half of pipe is constructed by hand as shown in Fig. 4, or by machine as shown in Fig. 8

Fig. 11 — Concrete is placed in hopper of pipe making machine by wheelbarrow. Workman has just placed two metal semicircular forms in position with the machine in motion. The workman is installing bracing for the last metal form. The duties of the workman standing on trench bank at left are to convey the metal semicircular forms and wood braces to the workman in the trench

crete to a minimum, compared to other types of mechanical monolithic concrete pipe making equipment, since less vibration is required for placing concrete in position for each individual half of the pipe.

3. The tendency for trench caving and sloughing is reduced due to less vibration of smaller equipment.

Some of the possible limiting features of this pipe construction method that should be noted are:

1. Due to vibration and movement of the pipe making machines in the trench during construction there is a tendency for the freshly placed concrete in the top of the pipe to slide off the oiled metal forms; to cause loose soil particles from the trench banks and walls to fall on the freshly constructed longitudinal seams; to render the trench walls vulnerable to caving and sloughing and to cause loose soil particles to fall on the freshly constructed longitudinal seams.

2. Machines and semicircular forms can only be used for one pipe size. Different pipe making machines and forms must be used for various pipe sizes.

3. There is a tendency for the wood spreader supports or footings placed in the bottom of the pipe during construction to sink or become embedded in the concrete.

4. It is difficult to satisfactorily install reinforcing steel in the pipe wall with this pipe construction method.

One-stage construction by machine method

This method of constructing monolithic cast-in-place concrete pipe involves the use of a machine that extrudes the completed concrete pipe directly in place in the trench in one complete operation. This machine in essence is a self-contained traveling form, it is relatively compact, and can be motivated by either electrical or mechanical equipment, or a combination of both. All features such as the engine, drum and cable mechanism, vibrators, and other accessories are contained within the unit. Fig. 12 to 14 illustrate the construction of one-stage monolithic concrete pipe with a self-contained traveling machine. This machine was developed about 1949.

Some of the desirable features of this method of constructing monolithic cast-in-place concrete pipe are:

1. The semicircular metal forms for forming the top inside portion of the pipe are positioned in place by a cylindrical guide and roller apparatus contained in the machine.

2. Apparatus can be installed in the machine for spraying oil on the metal forms as they are fed into position in the machine during pipe construction. This feature eliminates handling of the forms after oiling.

3. All controls for motivating the equipment are centralized within the machine and can be manipulated by a single operator.

Some of the possible limiting features of this pipe construction method that should be noted are:

1. Various mechanisms contained in the machine tend to make the equipment relatively expensive. The machine is heavy and tends to be awkward, particularly for the larger pipe sizes. The machine and metal semicircular forms can only be used for one pipe size. Different pipe making machines and forms must be used for various pipe sizes.

2. It is necessary for a workman to work in the inside rear portion of the machine during pipe construction. His duties are to fasten the metal semicircular forms as they pass through the machine, insert the spreader bars against these forms in the completed pipe as the machine moves forward, watch for imperfections in the concrete in the pipe bottom, repair any voids that appear in the concrete, and hand trowel the pipe bottom as required. These necessary activities within the pipe making machine could be termed as being performed under generally adverse or possibly hazardous conditions such as working in close quarters with possibility of excessive heat and inadequate air circulation, and coping with high noise levels from the engines, vibrators, spading mechanism, and other equipment.

3. Due to the vibration of the pipe making machine as it moves in the trench during construction, there is a tendency for the freshly placed concrete in the top portion of the pipe to slide off the oiled metal forms. This vibration and motion also tends to cause loose soil on the trench walls and bottom to slide ahead of the concrete in forming the bottom portion of the pipe. If excessively loose soil exists, this material has a tendency to accumulate thereby preventing concrete from being placed in the pipe bottom, resulting in a thin or possibly bottomless pipe. These various actions and motions of the pipe making equipment also tend to render the trench walls vulnerable to caving and sloughing.

4. The trench bottom must have a reasonably good vertical alignment to prevent the pipe making machine from "pitching" as the equipment moves forward during construction. When the rear end of the pipe making machine tips up, excessive concrete is emitted in forming the pipe bottom, thereby making the pipe wall excessively thick. Also, this excessive concrete accumulated in the pipe bottom has a tendency to float or lift the rear end of the pipe making machine. Likewise, when the rear of the machine tips down, insufficient concrete is deposited thereby making the wall of the pipe bottom of inadequate thickness. There is a tendency for the pipe making machine to pitch when operated at higher speeds.

5. There is a tendency for concrete aggregates to segregate as the material passes through the pipe making machine. Also, as the con-

Fig. 12 (above) — Front end view of one-stage monolithic cast-in-place concrete pipe making machine in position in trench. This machine is electrically operated. Drum and cable mechanism for pulling machine in the trench is attached at front end of the equipment. Metal semicircular forms are spaced on left trench bank for use as required as the operation moves forward. These forms are braced in position inside the machine by a workman. This is a traveling slip-form type of equipment with a semicircular bottom that conforms to the shape of the trench bottom

One-Stage Construction By Machine Method

Fig. 13 (above) — View of machine in trench constructing one-stage monolithic cast-in-place concrete pipe. This machine is of the type shown in Fig. 12. The completed monolithic concrete conduit is shown in foreground

Fig. 14 (left) — Interior view of completed monolithic cast-in-place concrete pipe constructed by one-stage machine method. The indentations visible in the upper portion of the pipe interior were made more prominent by vibration of the pipe making machine against the lapped upper forms during construction. Approximately two-thirds of the top portion of the pipe interior was formed by the metal semicircular forms. The bottom one-third portion of the pipe interior was formed by the semicircular trowel extending from the rear end of the pipe making machine. There is a distinct contrast in surface texture between the upper and lower portions of the pipe interior. The cross section of the pipe is termed as circular or near-circular

crete emits from the machine, the material in part slides along the trench bottom with tendency for the material to intermix with any existing loose soil.

6. A practical method of placing reinforcing steel in the pipe wall has not been developed to date with this pipe construction method.

SUMMARY

1. Monolithic cast-in-place concrete pipe is a low pressure head conduit, initially developed to replace various open or surface water carrier channels, canals, and ditches.

2. Several hundred miles of this type of conduit have been constructed in California since the late 1920's for irrigation and drainage uses and recently has been installed in other states.

3. In recent years this conduit has also been used for storm drains in various municipalities. Cognizance should be taken of the limitations of unreinforced monolithic concrete pipe for use in situations where risks from failure are high.

4. The limitations of this type of pipe should be carefully considered in selecting it for use. It is a serviceable and economical conduit where service conditions permit its use.

5. Proper trench and other conditions must exist for this type of pipe construction to be feasible and economical.

6. The hand method of monolithic cast-in-place concrete pipe construction is the method most extensively used to date, and is the only practical method in which reinforcing steel can be placed in the pipe wall.

ACKNOWLEDGMENT

Acknowledgment is made to many engineers, contractors, and others concerned with installation and use of this type of concrete conduit who through the years have generously provided information and opportunity for study and evaluation.

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Title No. 57-27

Simplifying Ultimate Flexural Theory by Maximizing the Moment of the Stress Block

By LYLE E. YOUNG

A method of simplifying ultimate flexural analysis is described. The stress block is defined by a method of maximizing the moment of the stress block about the neutral axis. Three separate stress functions are used to describe the stress in the concrete. The calculated type of failure and the ultimate moment are compared with results of reinforced concrete beam tests.

■ THE PREDICTION OF THE ULTIMATE moment that a beam can sustain has been dependent upon assumed stress distribution in the compression zone (stress block) and assumed values for the ultimate flexural strain in the concrete. Many shapes for the stress block have been proposed and described by a system of parameters or stress functions. The parameters and some portion of the stress function have been empirical in that they were dependent on beam tests for their determination. There can be little confidence in empirically derived expressions when there are large variations in concrete or steel properties or size of specimen.

It is the purpose of this paper to show that ultimate moment on a beam and the type of failure can be accurately predicted if the concrete and steel properties as described by stress-strain curves and the dimensions of the beam are known. This is done without dependence upon experimental beam tests.

Notation

A_s = area of tension reinforcing
b = width of rectangular member
d = distance from centroid of steel to compression edge of member
y = variable distance from neutral axis to a compression fiber
f = concrete stress at any distance y above the neutral axis
ε = concrete strain at any distance y above the neutral axis
f_c' = ultimate concrete stress
ε_c = concrete strain corresponding to f_c'
E_T = tangent modulus of elasticity of concrete
ε_u = ultimate concrete strain in flexure
f_{ys} = yield stress of the steel
ε_{ys} = strain corresponding to yield stress of the steel
E_s = modulus of elasticity of steel
ε_s = strain in the steel when the beam is at ultimate
T = total tension in the steel when the beam is at ultimate
e = natural logarithm base = 2.718
k = ratio of distance from top of beam to neutral axis to the depth of the beam
y_c = distance from neutral axis to the centroid of the stress block
C = total compressive force of stress block
f_a = average compressive stress of stress block
p = ratio of A_s to bd
M_u = ultimate moment

STRESS FUNCTIONS

Hognestad, Hanson, and McHenry[1] have shown that the stress-strain curves of concrete in compressive flexure and in direct compression show a striking similarity. However, the stress-strain curve of concrete in direct compression has a portion beyond the ultimate stress that is difficult to obtain in a testing machine due to the rapid release of energy stored in the parts of the testing machine as the specimen is loaded

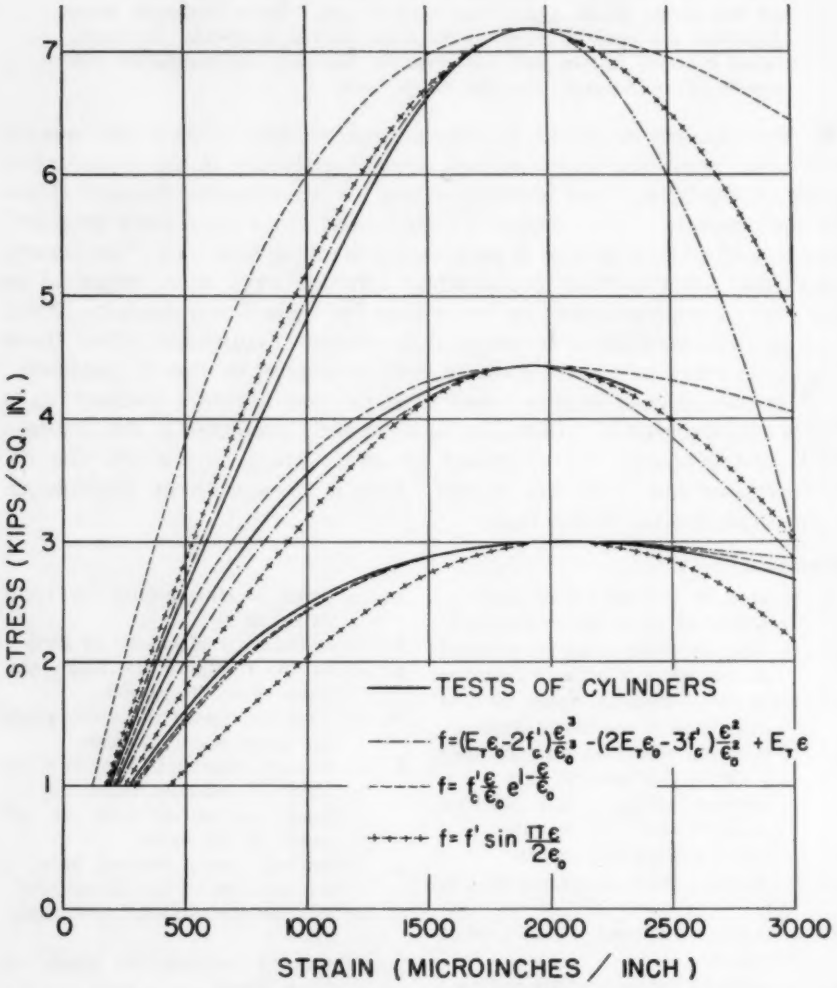

Fig. 1—Comparison of experimental stress-strain curves of three cylinders with curves described by stress functions

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to ultimate. This energy release also takes place in a beam test after ultimate load has been reached but with relatively smaller loads it is much less significant.

To approach this problem mathematically, three stress functions are used to describe the stress-strain curve of concrete:

$$f = (E_T\varepsilon_o - 2f_o')\frac{\varepsilon^3}{\varepsilon_o^3} - (2E_T\varepsilon_o - 3f_o')\frac{\varepsilon^2}{\varepsilon_o^2} + E_T\varepsilon \quad \text{Function 1}$$

$$f = f_o'\frac{\varepsilon}{\varepsilon_o}\, e^{(1-\varepsilon/\varepsilon_o)} \quad \text{Function 2}$$

$$f = f_o' \sin\frac{\pi\varepsilon}{2\varepsilon_o} \quad \text{Function 3}$$

In Fig. 1 curves described by these functions have been plotted along with the experimental stress-strain curves of three concrete cylinders. It can be seen that the functions approximate the actual stress-strain curves fairly well. In Fig. 2 the stress block and strain distribution at ultimate are shown for a rectangular beam cross section. The stress block is assumed to correspond in shape to the stress-strain curve of a cylinder up to some value of ultimate flexural strain. This ultimate strain is difficult to measure in a cylinder test but the following analysis eliminates the necessity of measuring this strain.

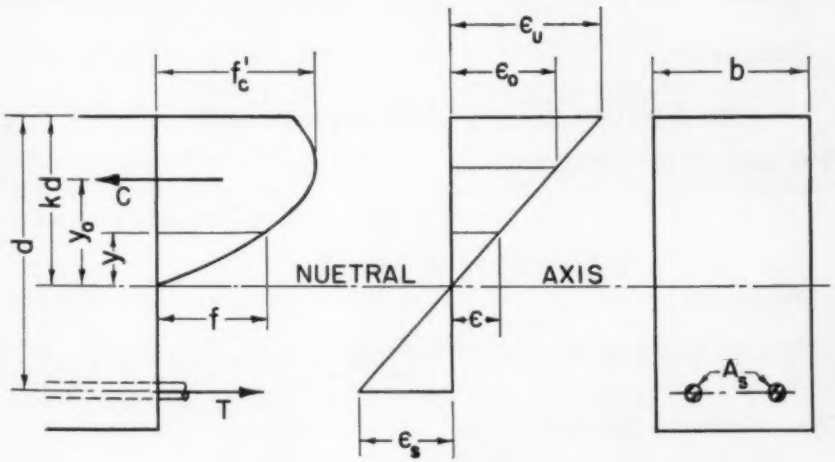

Fig. 2—Stress and strain conditions at ultimate load

DEFINING THE STRESS BLOCK

When a beam reaches ultimate load, the stress block will assume a shape as defined by the stress functions such that the resisting moment contributed by the total compressive forces about the neutral axis, y_oC, is a maximum.

$$y_oC = \int_0^{kd} y\, f\, b\, dy$$

From Fig. 2 by similar triangles: $y/kd = \varepsilon/\varepsilon_u$. Substituting in the expression for y_oC: $y = (\varepsilon/\varepsilon_u)kd$, $dy = (kd/\varepsilon_u)\,d\varepsilon$, ε_u for kd, and each of the three stress functions; integrating between limits and simplifying:

Function 1:

$$y_oC = b(kd)^2\left\{E_T\left(\frac{\varepsilon_u^3}{5\varepsilon_o^3} - \frac{\varepsilon_u^2}{2\varepsilon_o} + \frac{\varepsilon_u}{3}\right) + f_o'\left(\frac{2\varepsilon_u^3}{5\varepsilon_o^3} + \frac{3\varepsilon_u^2}{4\varepsilon_o^3}\right)\right\} \quad \text{(1a)}$$

Function 2:

$$y_oC = b(kd)^2 f_o' e\left\{e^{(-\varepsilon_u/\varepsilon_o)}\left(-1 - 2\frac{\varepsilon_o}{\varepsilon_u} - 2\frac{\varepsilon_o^2}{\varepsilon_u^2}\right) + 2\frac{\varepsilon_o^2}{\varepsilon_u^2}\right\} \quad \text{(1b)}$$

Function 3:

$$y_oC = b(kd)^2 f_o'\left(\frac{2\varepsilon_o}{\pi\varepsilon_u}\right)^2\left\{\sin\frac{\pi\varepsilon_u}{2\varepsilon_o} - \frac{\pi\varepsilon_u}{2\varepsilon_o}\cos\frac{\pi\varepsilon_u}{2\varepsilon_o}\right\} \quad \text{(1c)}$$

To determine ε_o when y_oC is a maximum, set $\partial(y_oC)/\partial\varepsilon_o = 0$ for Functions 1 and 2 and $[\partial(y_oC)]/[\partial(\pi\varepsilon_u/2\varepsilon_o)] = 0$ for Function 3:

Function 1:

$$\frac{\partial(y_oC)}{\partial\varepsilon_o} = b(kd)^2\left(\frac{E_T\varepsilon_o}{3} + f_o'\right)\left(-\frac{6\varepsilon_u^3}{5\varepsilon_o^4} + \frac{3\varepsilon_u^2}{2\varepsilon_o^3}\right) = 0$$

The solution of this equation is

$$\frac{\varepsilon_o}{\varepsilon_u} = 0.8 \quad \text{(2a)}$$

Function 2:

$$\frac{\partial(y_oC)}{\partial\varepsilon_o} = b(kd)^2 e f_o'\left[e^{(-\varepsilon_u/\varepsilon_o)}\left(\frac{\varepsilon_u}{\varepsilon_o^2} + \frac{4}{\varepsilon_u} + \frac{2}{\varepsilon_o} + \frac{4\varepsilon_o}{\varepsilon_u^2}\right) - \frac{4\varepsilon_o}{\varepsilon_u^2}\right] = 0$$

The solution of this equation is

$$\frac{\varepsilon_o}{\varepsilon_u} = 0.7 \quad \text{(2b)}$$

Function 3:

$$\frac{\partial(y_oC)}{\partial\left(\frac{\pi\varepsilon_u}{2\varepsilon_o}\right)} = b(kd)^2 f_c' \left\{ \frac{2\cos\frac{\pi\varepsilon_u}{2\varepsilon_o}}{\left(\frac{\pi\varepsilon_u}{2\varepsilon_o}\right)^2} - \frac{2\sin\frac{\pi\varepsilon_u}{2\varepsilon_o}}{\left(\frac{\pi\varepsilon_u}{2\varepsilon_o}\right)^3} + \frac{\sin\frac{\pi\varepsilon_u}{2\varepsilon_o}}{\frac{\pi\varepsilon_u}{2\varepsilon_o}} \right\} = 0$$

The solution of this equation is

$$\frac{\varepsilon_o}{\varepsilon_u} = 0.75 \qquad (2c)$$

Substitute the values for $\varepsilon_o/\varepsilon_u$ obtained above in Eq. (1a), (1b), and (1c):

Function 1: $y_oC = b(kd)^2 (0.026\, E_T\varepsilon_o + 0.392\, f_c')$

If $E_T = 1000\, f_c'$ and $\varepsilon_o = 0.002$

$$y_oC = 0.444\, b(kd)^2 f_c' \qquad (3a)$$

Function 2: $$y_oC = 0.472\, b(kd)^2 f_c' \qquad (3b)$$

Function 3: $$y_oC = 0.443\, b(kd)^2 f_c' \qquad (3c)$$

The evaluation of the total compressive force, C, follows: $C = \int_0^{kd} f\, b\, dy$ from Fig. 2. Substitute in this expression $kd\, d\varepsilon/\varepsilon_u$ for dy and ε_u for kd and each of the three stress functions and integrate. Evaluation of these expressions for values of $\varepsilon_o/\varepsilon_u$ as obtained in Eq. (2a), (2b), and (2c) results in:

Function 1: $C = bkd\,(0.071\, E_T\varepsilon_o + 0.586\, f_c') = bkdf_a$

where f_a = average stress
$= (0.071\, E_T\varepsilon_o + 0.586\, f_c')$ or if $E_T = 1000\, f_c'$ and $\varepsilon_o = 0.002$

$$f_a = 0.728\, f_c' \qquad (4a)$$

Function 2: $C = bkdf_a$ where $f_a = 0.797\, f_c'$ (4b)

Function 3: $C = bkdf_a$ where $f_a = 0.717\, f_c'$ (4c)

TYPE OF FAILURE AND LOCATION OF NEUTRAL AXIS

As a means of predicting the type of failure, concrete or steel, and locating the neutral axis at failure equate the total compressive force to the total tension force: $C = T$.

$$bkdf_a = A_sE_s\varepsilon_s \text{ (within the elastic range)} \qquad (5)$$

From the strain diagram of Fig. 2:

$$\frac{\varepsilon_u}{kd} = \frac{\varepsilon_s}{d - kd} \qquad (6)$$

Eliminate k between Eq. (5) and (6) and solve for ε_s:

$$\varepsilon_s = \frac{-\varepsilon_u + \sqrt{\varepsilon_u^2 + \frac{4f_a\varepsilon_u}{pE_s}}}{2} \qquad (7)$$

TABLE 1 — COMPARISON OF COMPUTED AND TEST VALUES*

Beam	b, in.	d, in.	$p = \frac{A_s}{bd}$	f_c', psi	f_{yp}, psi	Calculated type of failure using Function 1	2	3	Observed failure	Calculated ultimate moment, in.-lb. Using Function 1	2	3	Whitney[4] method	Actual ultimate moment in.-lb	Percent error Using Function 1	2	3	Whitney method
4308†	2.02	2.47	0.0442	3330	42,800	C	C	C	C	14,900	15,930	14,820	13,670	14,900	0	+ 6.9	− 0.5	− 8.3
4407	2.07	2.63	0.0367	4170	40,000	S	S	S	C	17,620	17,900	17,600	16,720	18,230	− 3.3	− 1.8	− 3.5	− 8.3
6304	2	2.45	0.0224	2760	75,800	C	C	C	C	10,930	11,860	10,900	11,030	11,480	− 4.8	+ 3.3	− 5.1	− 3.9
6306	2	2.47	0.0394	2760	75,200	C	C	C	C	12,320	13,250	12,570	13,300	12,100	+ 1.0	+ 9.5	+ 3.9	+10.0
6403	2	2.61	0.0151	4140	81,500	S	S	S	S	14,100	14,330	14,130	13,630	15,100	− 6.6	− 5.1	− 6.4	− 9.7
6406	2.01	2.47	0.0390	4190	75,200	C	C	C	C	17,460	18,740	17,520	17,180	16,300	+ 7.2	+15.0	+ 7.5	+ 5.4
6503	1.98	2.45	0.0227	4770	65,000	S	S	S	S	14,160	14,770	14,640	14,350	16,280	−13.0	− 9.3	−10.1	−11.9
213‡	5	5	0.0232	3060	55,200	Bal.	S	Bal.	C	126,600	126,300	124 200	120,500	127,500	− 0.1	− 0.9	− 2.6	− 5.5
221	5	5	0.0088	3060	53,400	S	S	S	S	54,700	54,300	54,400	53,700	48,700	+12.3	+11.5	+11.7	+10.3
222	5	5	0.0176	3060	53,400	S	S	S	S	99,700	100,000	98,700	96,500	95,000	+ 5.0	+ 5.3	+ 3.9	− 1.6
223	5	5	0.0264	3060	53,400	C	S	C	C	128,600	139,600	128,600	127,500	135,000	− 4.7	+ 3.4	− 4.7	− 5.6
224	5	5	0.0352	3060	53,400	C	C	C	C	135,400	143,500	135 400	127,500	132,000	+ 2.6	+ 8.7	+ 2.6	− 3.4
225	5	5	0.0440	3060	53,400	C	C	C	C	141,000	151,500	140,300	127,500	136,000	+ 3.7	+11.4	+ 3.2	− 6.2
321	5	5	0.0088	4750	53,400	S	S	S	S	55,800	56,500	56,400	55,500	55,200	+ 1.1	+ 2.4	+ 2.2	+ 0.5
341	5	5	0.0123	4750	48,100	S	S	S	S	68,700	69,100	69,200	68,700	69,800	− 1.6	− 1.0	− 0.9	− 1.6
421	5	5	0.0088	5870	53,400	S	S	S	S	59,200	59,300	59 200	56,200	54,600	+ 8.4	+ 8.6	+ 8.4	+ 2.9
441	5	5	0.0123	5870	48,100	S	S	S	S	69,800	71,100	70,700	69,600	66,700	+ 4.7	+ 6.6	+ 6.0	+ 4.4

*$E_s = 30 \times 10^6$ psi. ϵ_c assumed to be 2000×10^{-6} in. per in.
†Source for Beams 4308 to 6503 is Lash and Brison.[3]
‡Source for Beams 213 to 441 is Cox.[2]

From the solutions to Eq. (2a), (2b), and (2c) it is apparent that ε_u in Eq. (7) is equal to 1.25 ε_o for Function 1, 1.43 ε_o for Function 2, and 1.33 ε_o for Function 3.

If the value obtained for ε_s in Eq. (7) is less than ε_{yp} of the steel, a concrete failure will result. If the value for ε_s is greater than ε_{yp}, a steel failure will result. For a condition of balanced design $\varepsilon_s = \varepsilon_{yp}$.

To solve for k substitute the value of ε_s as found from Eq. (7) in Eq. (5):

$$k = \frac{pE_s\varepsilon_s}{f_o} \qquad \text{(concrete failure)} \qquad (8)$$

For values of ε_s greater than ε_{yp} this equation is not valid, but if the stress in the steel is assumed to remain constant after yielding:

$$k = \frac{pf_{yp}}{f_o} \qquad \text{(steel failure)} \qquad (9)$$

CALCULATION OF ULTIMATE MOMENT

From Fig. 2 it will be seen that:

$$M_u = C(d - kd + y_o) \qquad (10)$$

Expressions for C, k, and y_oC are given in Eq. (4), (8) or (9), and (3), respectively, in terms of the properties of the concrete and steel and the dimensions of the beam.

In Table 1 values of measured ultimate moment and observed type of failure from the experimental work of various investigators are compared with those obtained using the analysis described in this paper.

SUMMARY

It should be noted that once the initial mathematics has been performed, the resulting equations for determining type of failure, k, C, y_o, and M_u are of such simplicity that the investigation or design of beams using ultimate methods are no more complex than conventional elastic beam design. Note the following example:

Given: Concrete: $f_c' = 4000$ psi, $\varepsilon_o = 2000 \times 10^{-6}$ in. per in.

Steel: $f_{yp} = 40{,}000$ psi, $E_s = 30 \times 10^6$ psi

Beam section: $b = 4$ in., $d = 6$ in., $A_s = 0.409$ sq in., $p = 0.017$

Required: (a) Type of failure, and (b) ultimate moment

Equations: Use the average values obtained from the three stress functions: $\varepsilon_o/\varepsilon_u = 0.75$, $f_s = 0.747 f_c'$, $y_oC = 0.453\, b(kd)^2 f_c'$.

Type of failure [Eq. (7)]:

Substitute $\varepsilon_u = \frac{2000 \times 10^{-6}}{0.75}$, $f_c = 0.747(4000)$, $p = 0.017$, and $E_s = 30 \times 10^6$ in:

$$\varepsilon_s = \frac{-\varepsilon_u + \sqrt{\varepsilon_u^2 + \frac{4 f_c \varepsilon_u}{p E_s}}}{2} = 2840 \times 10^{-6} \text{ in. per in.}$$

$$\varepsilon_{ys} = \frac{f_{ys}}{E_s} = 1333 \times 10^{-6} \text{ in. per in.}$$

$\varepsilon_s > \varepsilon_{ys}$; therefore the beam is under-reinforced and a steel failure will result.

Location of neutral axis [Eq. (9)]:

$$k = \frac{p f_{ys}}{f_c} = 0.227 \qquad kd = 1.365$$

Total compressive force [Eq. (4)]:

$$C = bkdf_c = 4 \times 1.365 \times 0.747 \times 4000 = 16{,}320 \text{ lb}$$

Centroid of stress block [Eq. (3)]:

$$y_c C = 0.453\, b(kd)^2 f_c' = 0.453 \times 4 \times 1.365^2 \times 4000 = 13{,}530$$

$$y_c = \frac{13{,}530}{16{,}320} = 0.828 \text{ in.}$$

Ultimate moment [Eq. (10)]:

$$M_u = C(d - kd + y_c) = 16{,}320(6 - 1.365 + 0.828) = 89{,}200 \text{ in.-lb}$$

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Title No. 57-28

Reactivity of Ultrafine Powders Produced from Siliceous Rocks

By K. M. ALEXANDER

Siliceous materials such as quartz and basic or devitrified volcanic rocks, which are not likely sources of active pozzolan, become highly reactive when ground to ultrafine powders. If grinding is sufficiently prolonged an upper limit of activity is attained, beyond which continued increase in surface area does not produce any further general increase in pozzolanic reactivity. With widely differing types of siliceous material the upper limit of pozzolanic reactivity attained at very high surface areas tends to be the same in all cases, regardless of whether the mineral powders would be classed as pozzolanic, weakly pozzolanic, or nonpozzolanic when ground to the usual fineness specified for pozzolans. These observations can be explained by the presence of a disturbed layer of highly reactive material which is formed on the surface of siliceous mineral particles as a result of prolonged grinding.

■ THE POSSIBILITY THAT ALL SILICEOUS materials are capable of combining with calcium hydroxide, in the presence of water at normal atmospheric temperature, has been suggested by Davis,[1] who draws attention to the important role that fineness could play in this connection. If all siliceous materials are reactive in this sense then, in addition to the influence which this reactivity could have on the behavior of siliceous dusts which are in contact with hydrating portland cement, there may also be a significant effect on cement-aggregate bond, since reaction between the surface of siliceous aggregate pebbles and the portland cement matrix could have a considerable effect on the strength of the bond between the two. The ability of siliceous materials to develop pozzolanic properties when ground to ultrafine powders, and the implications which this behavior might have for the phenomenon of cement-aggregate bond formation were studied in the present experiments.

From the point of view of this investigation there were a number of specific questions to which answers were sought, and which can be defined as follows. With materials such as quartz, which are traditionally classified as nonpozzolans,[2] can ultrafine grinding produce a mineral powder which, when mixed with hydrated lime, at room temperature, will develop marked cementitious properties? If ultrafine

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grinding does in fact succeed in producing appreciable activity in siliceous minerals which would normally be regarded as inert, or at the best, feebly pozzolanic, it would be interesting to know if this induced activity could be raised, by further grinding, to a level comparable with that of known commercial pozzolans. If so, then does this induced reactivity go on increasing indefinitely with increasing fineness, or is a limiting value reached? If the latter alternative is correct, then is there a different limiting value for each type of siliceous material, or does there tend to be only one maximum level for induced pozzolanic activity, to which all highly siliceous materials approach at extremely high surface areas?

Finally, if all finely divided siliceous materials do have strongly developed pozzolanic properties, would quantitative estimates of their reactivities be helpful in, for example, predicting the strength of the bond which pebbles of such material could form with hydrated portland cement.

MATERIALS AND METHOD

Siliceous mineral powders for the present experiments were produced from the six aggregates described in Table 1, all of which were obtained from Victoria, Australia. An additional material was obtained from California Portland Cement Co., and consisted of calcined opaline shale from the Monterey formation in California. The hydrated lime and sodium hydroxide were of British Pharamacopea and commercial purity, respectively. The fine aggregate employed was the $-30+50$ mesh fraction of a round grained, washed quartz sand from Paringa, South Australia.

Final grinding of the mineral powders was done with rubber lined ball mills charged with hardened steel balls. Wet grinding was required for surface areas greater than 20,000 sq cm per g. Surface areas of less than 8000 sq cm per g were determined by the Blaine procedure,[3] while those from 8000 to 140,000 sq cm per g were measured by the method of Carmen and Malherbe.[4] All measurements were done at a porosity of 0.4.

Unless otherwise stated the mortars were of the following proportions by weight: powdered mineral/hydrated lime/sand, 2/1/3; water/(powdered mineral + hydrated lime), 0.28. Water-saturated mortars with this fixed water-cement ratio were produced by tamping until

water began to be expelled from the mix. Immediately after tamping the ½ x ½ x 4-in. specimens were extruded from the molds and stored in air filled, sealed tubes at 70 F for 7, 28, or 56 days. Both the moduli of rupture and the compressive strengths were measured, each result being the mean of six determinations. To avoid unnecessary duplication only one set of results will be reported since both lead to the same conclusions. The moduli of rupture will be given because they, rather than compressive strengths, are more closely related to one of the phenomena with which this investigation is concerned, namely, the force required to pull hardened cement off the surface of aggregate pebbles.

RESULTS AND DISCUSSION

Generally speaking, mineral powders produced from siliceous rocks fall into two groups, namely: (1) pozzolans, and (2) materials which for all practical purposes are inert or only weakly reactive when ground to a fineness within the range usually specified for pozzolans. In the following discussion the first point considered is whether ultrafine grinding can raise the activity of Group 2 materials to a level comparable with that of the pozzolans in Group 1.

Effect of ultrafine grinding on feebly pozzolanic and "inert" siliceous mineral powders

The weakly pozzolanic and "inert" siliceous rocks are represented here by Rock types P11 to P15 (Table 1). This group includes examples such as basalt, which is an inferior or wholly unsatisfactory pozzolan,[2] quartzite, which is usually described as being inert, and a number of volcanic rocks which, because they are for example, basic, or nonglassy, would not be considered as likely sources of highly active pozzolan. In Fig. 1 the 7-day and 56-day moduli of rupture of hydrated

TABLE I — DESCRIPTION OF AGGREGATES

Reference No.	Rock type	Location	Description
P11	Basalt	Bulla, Victoria, Australia	A Lower Pleistocene olivine basalt
P12	Woodendite	Woodend Victoria Australia	An unsuual basic glassy rock, somewhat similar to an absarokite. Tertiary (Pliocene) in age
P13	Trachyte	Mt. Macedon, Victoria Australia	A nonglassy trachyte containing porphyritic anorthoclase. Tertiary (Pliocene) in age
P14	Quartzite	Gelantipy, Victoria, Australia	A completely devitrified Mid-Devonian rhyo-dacite tuff in which bulk of silica is fine original quartz and secondary quartz
P15	Tuff	Murrindal River, Victoria, Australia	An acid tuff, probably Lower Devonian in age
V17	Opal	Butcher's Ridge, Victoria, Australia	Milky-white to brown common opal

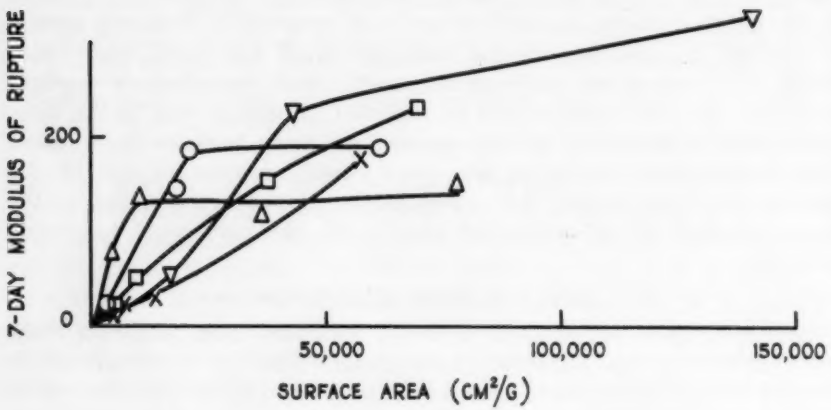

FIG. 1(a)

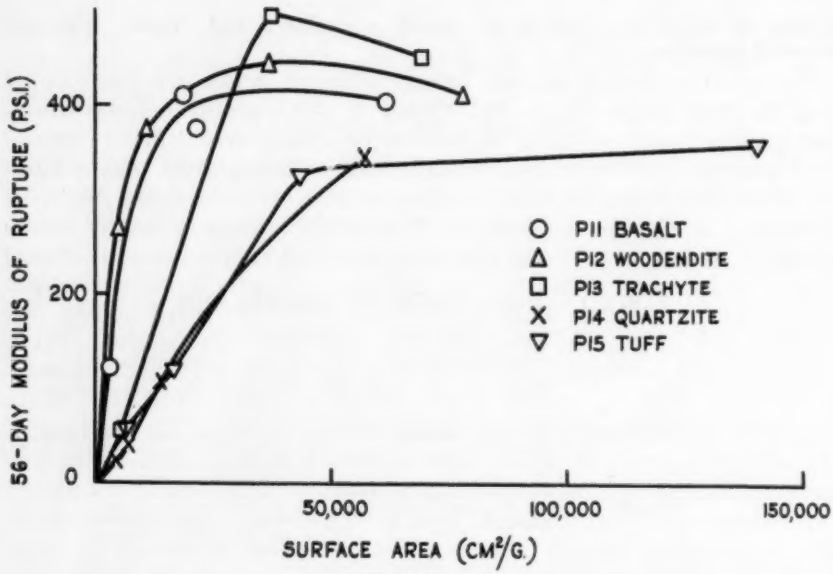

FIG. 1(b)

Fig. 1—Relationship between the fineness of siliceous mineral powders and the 7- and 56-day moduli of rupture of the hydrated lime mortars. Mortar composition: mineral powder/hydrated lime/sand, 2/1/3; water/(mineral powder + hydrated lime), 0.28, by weight. Curing temperature 70 F

lime mortars containing mineral powders from these rocks are plotted against the surface areas of the mineral powders. The main conclusions drawn from this data are as follows.

1. All the "nonpozzolanic" siliceous materials used here show marked pozzolanic activity at very high surface areas. For example, the 7-day moduli of rupture of lime mortars containing the present admixtures ground to 57,000 sq cm per g or more, are at least 150* psi while, in particular, the devitrified tuff P15, after a slow start in which a 7-day modulus of rupture of only 14 psi is attained at a fineness of 6000 sq cm per g, produces a 7-day modulus of rupture of 329 psi when the surface area is increased to 141,000 sq cm per g.

(2) The surface area at which a high level of reactivity is attained is very different for different materials. From Fig. 1b for example, it will be seen that in some cases (P11 and P12) a 56-day modulus of rupture of 400 psi is attained at 20,000 sq cm per g, while other materials (P14 and P15) do not come near to this level of activity until the surface area is of the order of 50,000 to 60,000 sq cm per g.

(3) With increasing fineness, reactivity tends to increase to a limiting value, usually before a surface area of 60,000 sq cm per g is attained. In the present experiments this effect was more pronounced at 56 days than at 7 days.

(4) Although some materials (e.g., P11 and P12) reach their limiting reactivity at considerably lower surface areas than others (e.g., P14† and P15), the limiting strength attained by the hydrated lime mortars after a few weeks of reaction shows relatively little dependence on the type of nonpozzolanic siliceous material used. Thus, for the five materials studied here, the mean 56-day modulus of rupture is 392‡ psi, with a coefficient of variation of 8.7 percent.

Comparison with active pozzolans

In Fig. 2 the quartzite, basalt, etc. described above are compared with known active pozzolans, represented here by calcined opaline shale from the Monterey formation, and by the opaline material V17 (Table 1) previously used by Vivian.[5] The comparison is made between the 7-, 28-, and 56-day moduli of rupture of hydrated lime mortars containing the admixtures in the form of ultrafine powders. The data show that at very high surface areas there is little to distinguish the pozzolans from their inert counterparts, even when the pozzolans, as well as the less active materials have been subjected to ultrafine grinding.

Some factors likely to affect the strength of lime mortars containing ultrafine siliceous mineral powders

Before leaving the subject of the reactivity of so-called inert or weakly-reactive siliceous minerals there are two further questions to be answered. Firstly, are the ultrafine powders more strongly pozzo-

*The corresponding compressive strengths attained were all greater than 1000 psi at 7 days.
†By 56 days (Fig. 1b) quartzite P14 had still not attained limiting strength at the maximum surface area (57,650 sq cm per g) achieved here, but the material is not expected to behave very differently from P15 in which the principal "pozzolanic" component is the same, i.e., quartz.
‡The corresponding mean compressive strength was 3100 psi with a coefficient of variation of 7.4 percent.

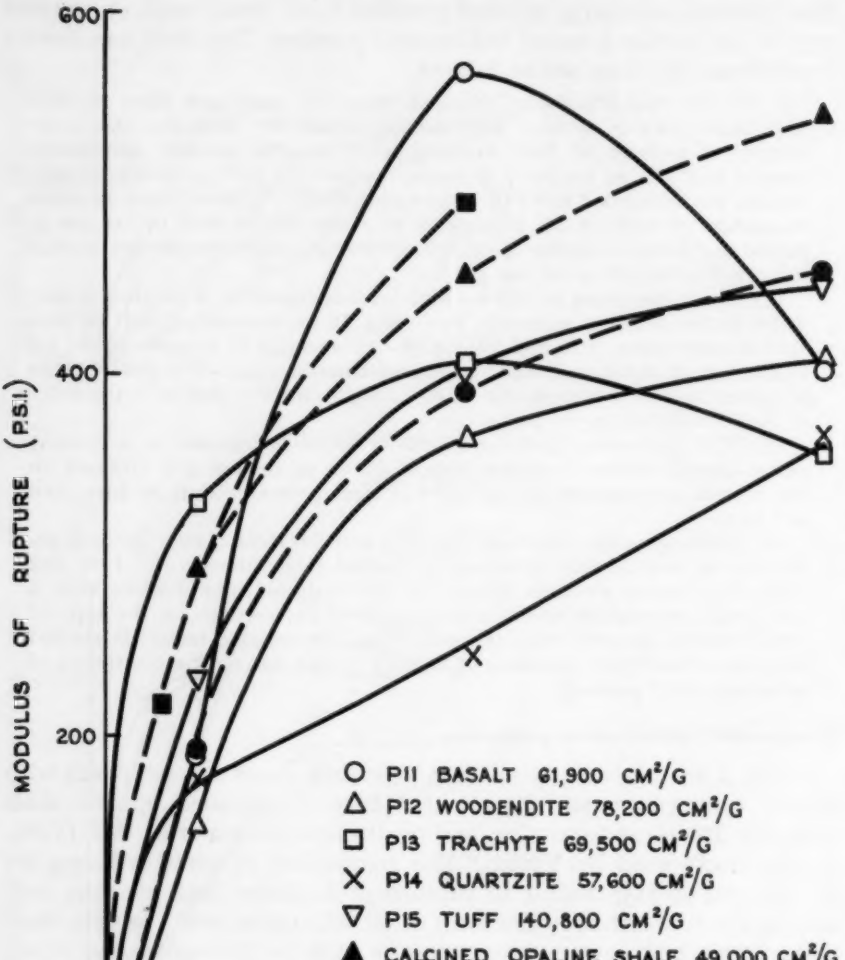

Fig. 2—Comparison between the moduli of rupture of lime mortars containing ultrafine pozzolans and ultrafine "inert" siliceous mineral powders, at ages of 4, 7, 28, and 56 days. Mortar composition: mineral powder (or pozzolan)/hydrated lime/sand, 2/1/3; water/(mineral powder or pozzolan + lime), 0.28, by weight. Cured at 70 F

lanic at some other ratio of hydrated lime to powdered mineral? Secondly, since quantities of alkali comparable with those present in portland cement can greatly affect the strength developed by lime mortars containing highly active pozzolans,* will alkali have an analogous effect on the strength of lime mortars containing ultrafine mineral powders prepared from less reactive siliceous rocks?

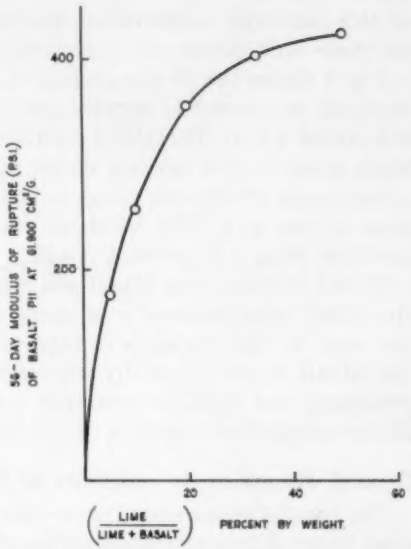

Fig. 3—Relationship between 56-day modulus of rupture and the composition of a P11 powdered basalt-hydrated lime cement. Fineness of basalt P11: 61,950 sq cm per g. Mortar composition: (powdered basalt + hydrated lime)/sand, 1/1; water/(powdered basalt + hydrated lime), 0.28, by weight. Cured at 70 F

Dependence of strength on proportion of hydrated lime—The 56-day moduli of rupture were determined for mortars in which 5, 10, 20, 33, and 50 percent, by weight, of the pozzolan-lime cement, consisted of hydrated lime. The mineral powder employed was the 61,900 sq cm per g basalt previously used to obtain the highest surface area point for P11 (Fig. 1). The results (Fig. 3) suggest that strength increases to be expected by using more than 33 percent of hydrated lime are slight. Increasing the hydrated lime content to 50 percent, for example, raises the modulus of rupture by only 5 percent.

It is of interest to note that very little hydrated lime is required to produce marked cementitious properties. Thus, the 56-day modulus of rupture of the mix with only 5 percent of hydrated lime present is 44 percent of the value attained at the 2/1 pozzolan-hydrated lime ratio used throughout the present investigation.

Effect of alkali—In these experiments with the inert and less reactive minerals P11 to P15, no attempt was made to use alkali for activating relatively coarse materials (e.g., 3000 to 8000 sq cm per g surface area). Instead, the mineral powders used were those whose activity had been raised, by ultrafine grinding, to the highest level attained in the present investigation. Since the materials were already capable of developing cementitious properties in the presence of hydrated lime, the object

*It was shown previously[6] that as little as 0.3 percent Na_2O (by weight of pozzolan + calcium hydroxide) added to the mixing water increased the 7-day and 6-month moduli of rupture of lime mortars containing V17 by 112 and 55 percent respectively.

of this particular experiment was to see if alkali had a significant effect on these well developed pozzolanic characteristics.

Fig. 4 shows the 28-day moduli of rupture of the alkali-containing lime mortars, expressed as percentages of the strengths of the mortars without added alkali. The alkali additions, in the form of sodium hydroxide, were made to the mixing water, and the amounts added are given as percentages of the combined weight of powdered mineral and hydrated lime in the mix. The batch of lime used contained 0.01 percent K_2O and less than 0.01 percent Na_2O.

It will be seen that alkali always affected lime mortar strength, that the effect was positive with some minerals, negative with others, that the sign of the strength change produced by progressive increases in the alkali content usually reversed as high alkali contents were approached, and that, for example, 0.6 percent of alkali could change the 28-day modulus of rupture of the lime mortar by as much as 50 percent.

General discussion on reactivity of finely divided siliceous materials

The results given above show that at very high surface areas even the most inert siliceous materials acquire reactivity comparable with that of conventional pozzolans and that the degree of reactivity does not increase indefinitely with fineness, but reaches a steady limiting value which is roughly the same for all the materials investigated here. The following explanation of this behavior is suggested by the known properties of siliceous dusts.

For the purpose of the present hypothesis the characteristics of quartzite P14 will be considered in some detail both because it is the most inert of the materials studied and because a good deal is known about the properties of quartz dusts.

Dempster and Ritchie[7,8] have found that grinding produces a disturbed layer on the surface of siliceous dust particles. The layer is highly soluble, dissolving in, for example, borate buffer at pH 7.5, and can be regenerated by further grinding or by agitation with abrasive powders. The layer thickness attains a fairly constant maximum value after a grinding time probably much less than, and certainly not more than 1 or 2 hr, and is not significantly increased in thickness by subsequent grinding. Working with quartz dusts and using both density measurement and differential thermal analysis, it was shown that the layer had a total thickness of 0.11 to 0.15 microns, and it was pointed out that prolonged grinding should produce a dust with a mean particle size less than twice the thickness of the disturbed layer.

These same workers obtained additional evidence from solubility measurements and from electron diffraction studies which suggested that the disturbed layer has a graded structure. There is a highly soluble surface layer 0.02 to 0.03 microns thick which is a vitreous skin produced by surface flow during grinding and which blends, via a

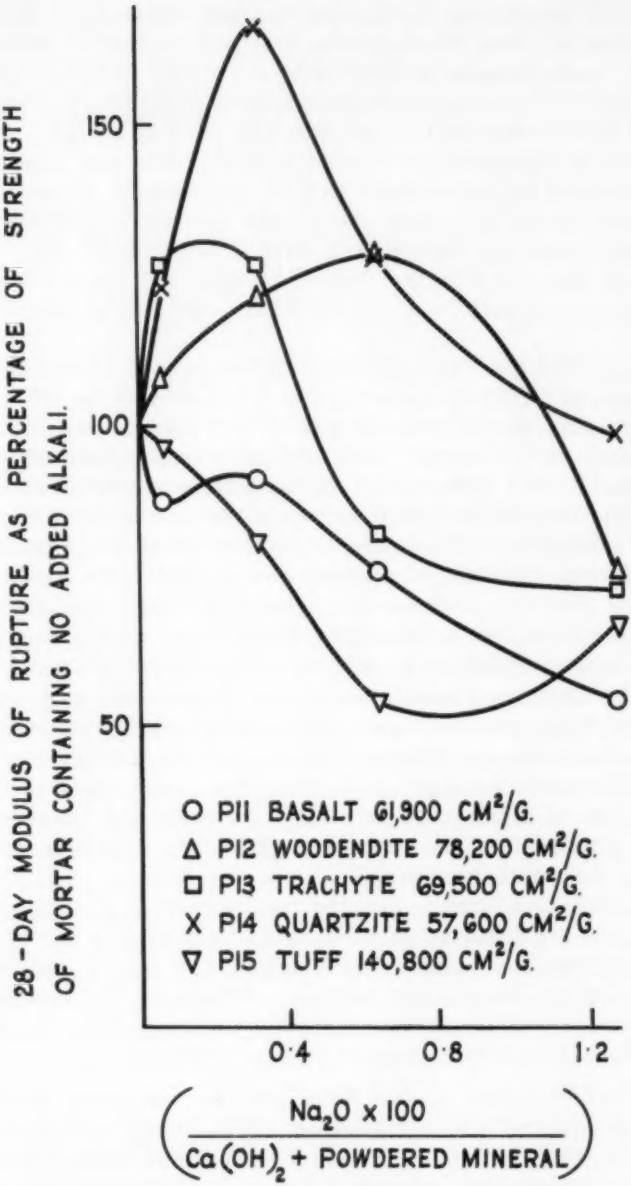
28-DAY MODULUS OF RUPTURE AS PERCENTAGE OF STRENGTH OF MORTAR CONTAINING NO ADDED ALKALI.
150
100
50
O P11 BASALT 61,900 CM²/G.
Δ P12 WOODENDITE 78,200 CM²/G.
□ P13 TRACHYTE 69,500 CM²/G.
X P14 QUARTZITE 57,600 CM²/G.
▽ P15 TUFF 140,800 CM²/G.
0·4
0·8
1·2
(Na2O x 100 / Ca(OH)2 + POWDERED MINERAL)

Fig. 4—Effect of sodium hydroxide on the 28-day moduli of rupture of hydrated lime mortars containing ultrafine mineral powders. Mortar composition: powdered mineral/hydrated lime/sand, 2/1/3; water/(powdered mineral + hydrated lime), 0.28, by weight. Cured at 70 F

less soluble transitional layer, into the crystalline core. Independent confirmation of these findings was obtained by Smith[9] who used reflectance measurements to show that a complex surface layer of approximately 0.12 microns thickness was present on finely ground quartz and that the extreme outer layer was 0.02 microns thick.

The data of Dempster and Ritchie, and of Smith, thus suggest that if quartz is milled for more than 1 or 2 hr, any particles of approximately 0.3 microns diameter or less will consist entirely of altered material. Independent work by Gordon and Harris,[10] who used x-ray diffraction techniques, leads to a larger value (0.5 microns) for the diameter of the largest quartz particle which has a disturbed crystal lattice throughout.

The finest P14 quartzite dust used in the present experiments had a surface area of 57,650 sq cm per g, and was produced by 100 hr of milling. Information on the particle size of this material was obtained by sedimentation experiments, with the microscope, from surface area measurements, and from x-ray diffractometer studies. Sedimentation experiments showed that the diameter of the smallest particles present was approximately 0.15 microns, while microscopical examination revealed relatively few particles larger than 1 micron. On the simplifying assumption that the particles are spheres of equal diameter, the air permeability data give a mean particle diameter of 0.39 microns. This latter value was confirmed by measuring the diffracted intensity of the $11\overline{2}2$ line of ultrafine P14 relative to that of P14 with a mean particle diameter of 5 microns, and applying the relationships between relative diffracted intensity and effective particle diameter as reported by Gordon and Harris.[10] The conclusion to be drawn therefore is that with the exception of a few relatively large particles, the finest sample of quartzite P14 used in the present experiments would be completely altered by the grinding process.

If the assumption is made that the reactivity of finely divided quartz is confined to the disturbed layer formed on the surface of the particles during grinding, any undisturbed inner core of quartz remaining unreactive, then the relationship between surface area and reactivity, and the fact that reactivity attains a limiting value instead of increasing indefinitely with surface area, can be explained.

Relationship between surface area and the reactivity produced by ultrafine grinding of inert siliceous materials—Using surface area measurements as the basis for calculating mean particle diameters, and thus making the considerable simplification that all particles are spherical and of equal diameter, an estimate can be obtained of the amount of disturbed material present in the sample, since the depth of the altered layer is independent of grinding time after the first hour. In Fig. 5 the weight of disturbed material per unit weight of quartz originally

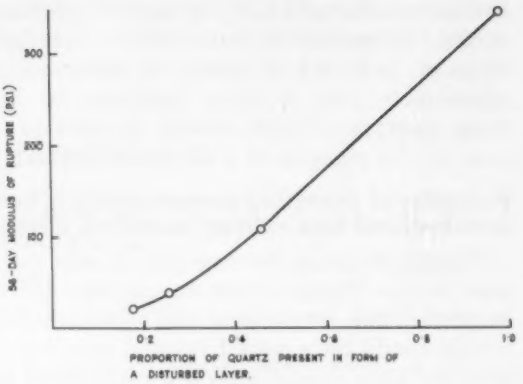

Fig. 5—Dependence of lime mortar strength on proportion of quartz present in the form of a disturbed layer on the particle surface. Mortar composition: powdered quartz/hydrated lime/sand, 2/1/3. Water/(powdered quartz + hydrated lime), 0.28, by weight. Cured at 70 F

present is plotted against the 56-day modulus of rupture of the lime mortar. The calculations are based on a value of 0.15 microns for the thickness of the reactive layer. It will be seen that about 20 percent of the silica must be in the activated state before appreciable strength is developed, and that thereafter the strength of the lime mortar increases almost linearly with the proportion of quartz converted to active material by grinding.

Reason for an upper limit to the pozzolanic activity induced by ultrafine grinding—Fig 5 shows that 98 percent of quartzite P14 was converted to active silica at the maximum surface area attained here. No further general increase in reactivity would be expected if grinding were continued. This is confirmed by the experiments with the devitrified tuff P15 which consists largely of fine original quartz and secondary quartz. Although P15 was eventually ground to 140,000 sq cm per g, the 56-day modulus of rupture (Fig. 1b) of the hydrated lime mortar did not significantly exceed that of the P14 quartzite ground to slightly less than 60,000 sq cm per g.

It should be pointed out that with highly reactive pozzolans such as opal, and with siliceous minerals of intermediate reactivity, the relationship between fineness and reactivity will differ from that described for inert siliceous minerals such as quartz. Opal and similarly reactive materials are capable of reacting in depth and do not require a surface layer of disturbed material to achieve a high level of activity. To ensure reasonably complete reaction of these more active materials it is not necessary therefore to reduce the mean particle diameter to a size equivalent to twice the thickness of a disturbed surface layer. The grinding, relatively coarse by the present standards, which active materials receive when used as commercial pozzolans, merely enables the admixture to be dispersed efficiently throughout the cementitious matrix, and helps to increase the rate at which the pozzolan reacts rather than

the extent to which it is capable of reacting after long periods of curing. Materials of intermediate reactivity represent a transitional stage of behavior in which a disturbed layer could account for an appreciable part of their reactivity at high surface areas, although these materials would already be able to react to a significant extent even in the absence of a disturbed surface layer.

Possibility of predicting cement-aggregate bond strength from experiments with hydrated lime mortars containing ultrafine siliceous mineral powders

Finally, there is the question of whether information obtained from lime mortar strength tests on ultrafine siliceous minerals could be used to predict the strength of the bond which pebbles of these same materials would form with hydrated portland cement. It might be thought that the lime mortar strength tests could be useful in this respect because it is possible that pozzolanic reaction between the surface of a siliceous aggregate pebble and the lime released during the hydration of portland cement may make a significant contribution to cement-aggregate bond strength.

However, there are several reasons for regarding the lime mortar test as unsatisfactory for predicting bond strength. By comparison with direct measurements of cement-aggregate bond strength[11] it offers no saving in time or labor because long grinding times are required to obtain reactive powders from many of the nonreactive siliceous minerals. The lime mortar strength test does not even have the advantage of producing strengths similar in magnitude to cement-aggregate bond strength.[11] Two further disadvantages are that the lime mortar test does not take into account any effect, on bond, of surface texture of the aggregate and in any case the measure of activity obtained is particularly sensitive to the presence of disturbed surface layers generated on the surface of the powdered mineral by the process of grinding, which tends, for example, to produce the same lime mortar strength from all ultrafine mineral powders with a high silica content.

CONCLUSIONS

(1) Siliceous materials such as quartz and basic or devitrified volcanic rocks, which are not regarded as likely sources of active pozzolan, become highly reactive when ground to ultrafine powders.

(2) If grinding is sufficiently prolonged an upper limit of activity is attained, beyond which continued increase in surface area does not produce any further general increase in pozzolanic reactivity.

(3) With widely differing types of siliceous material the upper limit of pozzolanic reactivity attained at very high surface areas tends to be the same in all cases, regardless of whether the mineral powders would be classed as pozzolanic, weakly pozzolanic, or inert, when ground to the usual fineness specified for pozzolans.

(4) The observations stated in Conclusions 1 to 3 can be explained by the presence of a disturbed layer of highly reactive material which is formed on the surface of siliceous mineral particles as a result of prolonged grinding.

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CONCRETE BRIEFS

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Sporting Club of Portugal Stadium

By RUY JOSÉ GOMES*

THE FACILITIES AND STANDS of the stadium for the Sporting Club of Portugal, Lisbon (JOURNAL cover) are arranged along an approximately oval ring, forming a strip of about 33 m mean width, around the playing grounds. With a perimeter of 634 m, the stadium occupies an area of 20,300 sq m. The external ring-shaped strip of this area, 22 m wide, corresponding to an area of 13,300 sq m is occupied by the superstructure of the stadium. This consists of portal frames in a radial position (stringer frames) which carry at different levels the roof, upper stands, tribunes and boxes, gangways, and service facilities for the field and athletes. The inside ring, with variable width, corresponds to an area of about 7000 sq m and is occupied by stands.

DESIGN OF STADIUM

Stringer frames of the stadium superstructure

The essential members of the stadium superstructure are the stringer frames (Fig. 1) arranged in a radial position around the playing ground.

The distance between the frames was based on economy, being a compromise between the advantage of reducing, as far as possible, the number of frames and the need to ensure acceptable spans for the structures they support, taking into account the loads acting on the structure. A uniform interval of about 6 m between frames, measured along the outside perimeter of the stadium was adopted, defining 112 radial moduli between the stringer frames.

The shape of the frames was chosen in accordance with the arrangement of the members they support. The frames at the ends and one side of the field—amounting to about three-quarters of the stadium perimeter—are totally occupied by the stands and consist of (Fig. 2): an upper inclined beam with the slope of the stands and prolonged to the outside of the field by a horizontal cantilever; an intermediate beam at the level of a gallery under the stands; a lower horizontal beam at the level of the crowning of the foundations, under the floor of the lower gallery where the service facil-

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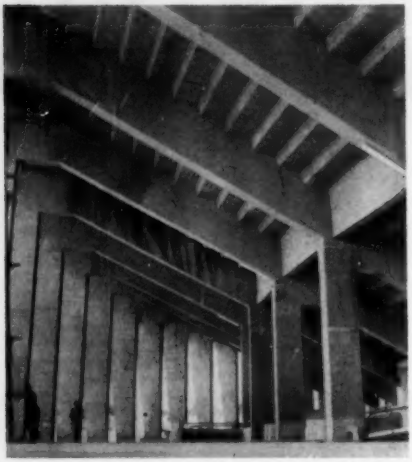

Fig. 1—View of stadium from beneath the stands

ities of the fields and athletes are located; three columns, spaced about 10 m.

In the frames of the west stands, where the tribunes and boxes are located, the upper inclined beam has a horizontal stretch corresponding to the space occupied by these. In this case, the intermediate horizontal beam is above the general gangway and carries a small intermediate gallery. The lower beam corresponds to the general gangway, as in this stand no field facilities are installed under the gallery. These frames have but two columns.

The structural frames, closed at the base by the lower beam, have simple hinged supports bearing on the foundations.

Bracing between frames is achieved by the members they support: stand steps, gangways, and the earth-retaining wall, located in the vertical of the internal columns of the frames.

Gangways, tribunes, and boxes

The current structures of the gangways were designed as solid slabs, with crosswise reinforcement, in panels limited by the beams of the frames and the longitudinal beams supported by these. This solution was adopted since the repetition of equal panels made it economically feasible to adopt standardized, reusable forming; at the same time, it was sought to use the bracing effect of the solid slab.

The structures of the tribunes and boxes, according to the design, consist of horizontal slabs, vertical beam walls running lengthwise between the frames, and the beams complementary of these.

Stand steps

When designing the stand steps, the following facts were taken into account: (1) The most logical cross section of the steps would be an L-shape, with the vertical wing as the bend-resisting member. (2) The steps are repeated along the 112 radial moduli. (3) The direct exposure of the stand to solar radiation and the external temperature variations requires its separation into sections, by joints not too far apart. (4) The concreting *in situ* over stepped formwork would be difficult and expensive and would not allow a satisfactory consolidation and finishing of the steps.

The preceding analysis showed that the best solution would be to adopt stands consisting of prefabricated steps, placed and juxtaposed at the site. This solution ensures the discontinuity of the stand surfaces and makes it possible to cast the steps in rigid and satisfactory forms and to use efficient consolidating devices.

Prefabrication is particularly advantageous if the number of different shaped members can be reduced to a minimum. In the present case, the differences in dimensions between the steps were due to the different heights of the successive tiers of seats — on account of the parabolic shape of the stand, adopted for reasons of visibility—and to the different lengths of each row, due to the trapezoidal shape of the stretches between the frames.

The differences in height can be solved by means of an adjustable bottom form. To eliminate the differences in length, the upper beams of the frames have a horizontal flange of

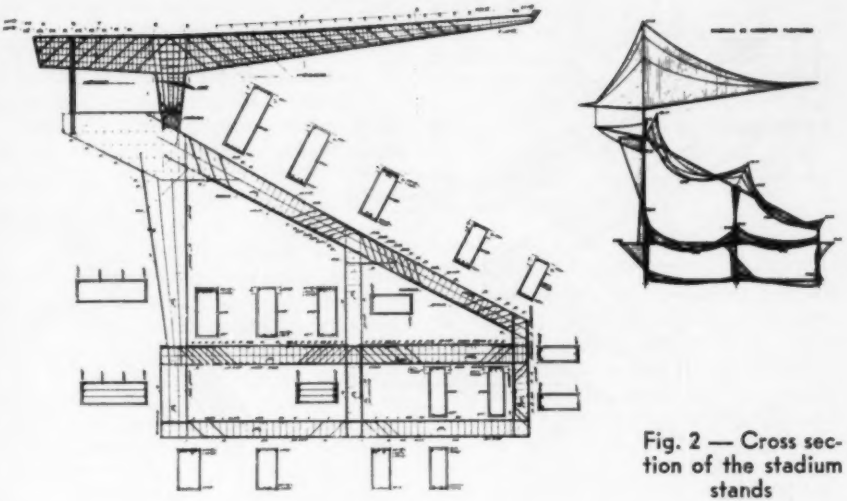

Fig. 2 — Cross section of the stadium stands

trapezoidal shape, forming parallel supports for the steps; the transition stretches of the different panels of the steps would be cast in place on these widened flanges of the beams.

The weight of the prefabricated units, about 1600 kg per unit, was considered acceptable for the mechanical handling equipment likely to be available at the job site.

Roof over stands

The stands were to be covered by a roof for the perimeter of the stadium. The width of the roof was designed to be about ⅔ of the mean width of the stands.

To ensure a good visibility in all the seats, the roof was a canopy with a span of 22 m, consisting of cantilevers, one for each stringer frame and supported by it. The canopy support was aligned with the higher tiers.

The cantilevers were assumed to be statically determinate members, simply supported at the frames. They comprised a front stretch, 22 m long; a back counterweight arm, 8 m long; a column at the junction of the two preceding members, carried, by means of a hinge, by the top of the external column of the frame; a tie at the back end portion of the counterweight, anchoring this at the external projecting point of the frame below; a counterweight consisting of a ballast box at the top of the back arm of the cantilever.

After choosing the shape of these load-bearing members, the structure of the roof was studied. The problem to be solved was that of a cantilever of large span—exceeding the spans of the majority of stadium roofs already built —in which the lightness of the material was essential. Additionally, its moderate slope and its direct exposure to weathering and to alternate thermal stresses had to be taken into account.

After studying the solutions adopted in similar roofs, a different solution was suggested: light aluminum sheets. This material, with a weight of about 3 kg per sq m (to which the weight of the metallic structure, estimated at about 10 kg per sq m, has to be added) makes it possible to obtain a roof about 20 times lighter than any other of the usual or possible solutions using reinforced concrete.

A study (by means of summary design methods) of the comparative costs of a concrete shell roof and structural cantilevers or an aluminum roof hang-

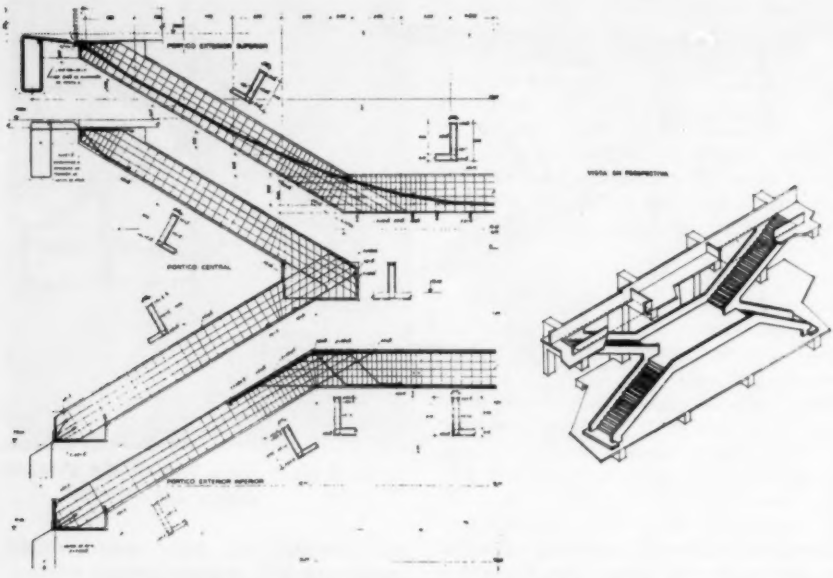

Fig. 3—Cross section and perspective of tribune stairs

ing from a metallic structure supported by the cantilevers, showed* that the aluminum roof might be the most favorable solution.

The constructional aspects of this roof were also studied. From the point of view of durability and maintenance, an aluminum roof has an excellent resistance to corrosion and the discontinuity required to withstand alternate thermal stresses. On the other hand, monolithic concrete has low resistance to thermal stresses and requires maintenance of the waterproofing used to seal eventual cracks due to those stresses.

Tribune stairs

Among the minor structures of the stadium, the tribune stairs deserve special mention (Fig. 3). It is a stair with two flights symmetrical in relation to an intermediate landing. The structure consists of lateral portal frames, trapezoidal in shape, and central portal frames with the shape of a lying V.

The lower lateral trapezoidal frame resists as a polygonal arch, spanning about 18 m. The upper lateral frame hangs from the stringers, behaving as a funicular with bars subjected to bending stresses, tension being predominant; a rational arrangement of the reinforcements was adopted in this member, corresponding to the shape of the pressure line for the most likely loads.

The upper supports of the central frames hang from the stringers and the lower are supported; the upper beam resists to bending and tension and the lower withstands bending and compressive stresses.

The steps and the landing slab are the connecting members between the frames making up the structure of the stair.

*See Acerca de Coberturas em Consola de Grande Vao," *Arquitectura*, No. 61, Dec. 1957.

ALTERNATIVE METHODS ADOPTED IN CONSTRUCTION

In the proposals for construction or after the construction had begun, the contracting firm suggested some alternative execution methods which were approved.

Gangway floors

Instead of the reinforced concrete solid slabs foreseen in the design, the gangway floors were made of prefabricated slabs made up of prestressed joists, hollow block between the joists, and a concrete compressive topping. The reduction of the bracing effect of these slabs as compared with the solid slabs foreseen in the design was compensated for by the greater number of girders between the stringer frames.

Confirming the technical objections presented against the adoption of this alternative, cracks have appeared at the supports of the slab panels 1 year after the completion of the structure and the wear resistance of the concrete topping has been low.

Lower stand ring, near the cycle tracks

According to the design, the lower stand ring was to be supported directly by a compacted embankment but an alternative was presented and built, consisting in steps supported by stone masonry radial walls.

This alternative had the advantage of not requiring a perfect earth compaction and consequent delays, and eliminated the risk of the heterogeneous soils available for the embankment.

Stand steps

Instead of the foreseen solution of prefabricating the stand steps, these were cast in place with resulting lower wear resistance and unsatisfactory finishing. The contracting firm sought by this means to eliminate the use of lifting and conveying equipment to lay the prefabricated steps.

Stand roof

During the construction, an alternative roof system presented by the contracting firm was approved and built. This system, while maintaining the structure indicated in the design—cantilevers with back counterweight arm with a ballast box, central hinged column, and back anchoring tie—replaced the aluminum sheet covering with reinforced concrete shells and the reinforced concrete cantilevers by prestressed concrete cantilevers, following in the latter case a suggestion of the design.

The economic feasibility of this alternative was due to exceptional conditions in the supply of materials and prestressing equipment, of which the contracting firm could take full advantage.

Credits

The stadium of the Sporting Club of Portugal was built according to the architectural design of Anselmo Fernandes and Sá da Costa, architects, and the structural design of the author. The contractor was the firm Alves Ribeiro; Mário Barata, civil engineer, was the resident engineer for the owner.

The Problems and Practices section (which appears in Concrete Briefs section as space allows) is an open forum on everyday topics. JOURNAL readers are invited to contribute their solutions to problems that occur in the field or office—whether common or unusual.

Items appearing in this section can vary from 100 to 1000 words. It is an ideal place for those practical items generally considered too short to be be submitted for publication, but containing material of potential importance to a great many.

Pilot Bond Tests of Large Reinforcing Bars

By FERDINAND S. ROSTÁSY and EIVIND HOGNESTAD*

The special large sizes of reinforcing bars designated as #14S and #18S, in ASTM A 408-58T, were principally manufactured for use as longitudinal reinforcement in columns. In recent years, however, such bars have also been used as reinforcement in large flexural members. Since the allowable bond stresses in the ACI Building Code are primarily based on tests performed with small sized bars,[1] a need for test data regarding bond properties of large bars has therefore been felt.

At the request of George F. Long, chairman, Committee on Reinforced Concrete Research, American Iron and Steel Institute, the PCA Laboratories carried out a pilot series of pull-out bond tests during the fall of 1959.

TEST SPECIMENS AND PROCEDURE

Table 1 shows that #10 bars were included in this pilot series. This was done to permit comparison with the results of the extensive tests of Clark[1] in which the #10 bar was the largest included. The #10 bars were pulled out of 9 x 9-in. prisms with embedment lengths of 12 and 16 in., as they were in Clark's tests. While in Clark's pull-out tests the bars were located eccentrically with 2 in. cover and with a reinforcing spiral around them to prevent premature splitting, the bars in the tests reported here were placed concentrically without any transverse reinforcement at all. The prism size and embedment lengths for the #14S and #18S bars were scaled up in proportion to the bar diameter as shown in Table 2. The mechanical and geometrical properties of the bars used are given in Table 1. The "Hi-Bond" bar, identified as bar type No. 8 in Clark's paper, was used throughout.

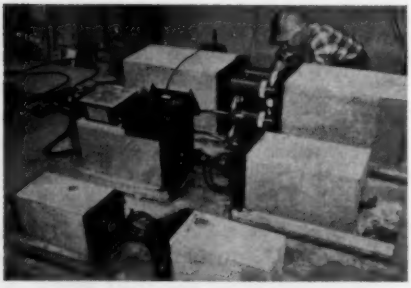

Fig. 1—Measuring and testing setup

Clark's bond-slip curves for the #10 bar are given for an average concrete strength of 3500 psi at the time of testing. In the present tests, a strength of 3500 psi was sought at a testing age of 7 days. The cement used was a blend of Type I cements. The aggregates used were Elgin sand and gravel of 1½ maximum size. Concrete was proportioned by weight in a cement-sand-gravel ratio of 1:3.31:5.42. The water-cement ratio, by weight, was 0.66. Specimens were cast in their final testing position. The concrete was compacted by an internal spud vibrator, and the surface was later troweled. The specimens and 6 x 12-in. cylinders were cured moist for 5 days; forms were stripped at 3 days. For the last 2 days prior to testing at 7 days, the specimens and cylinders were stored at 70 F and 50 percent relative humidity. Concrete strengths are listed in Table 2.

Large forces and heavy specimens are involved for the large reinforcing bars. The customary method for pull-out testing by means of conventional testing machines was, therefore, considered too cumbersome, and the test setup shown in Fig. 1 was chosen. Each pull-out specimen consisted of two prisms cast at each bar end. The dimensions are listed in Table 2. The

*Associate Development Engineer and Manager, respectively, Structural Development Section, Portland Cement Association, Research and Development Laboratories, Skokie, Ill.

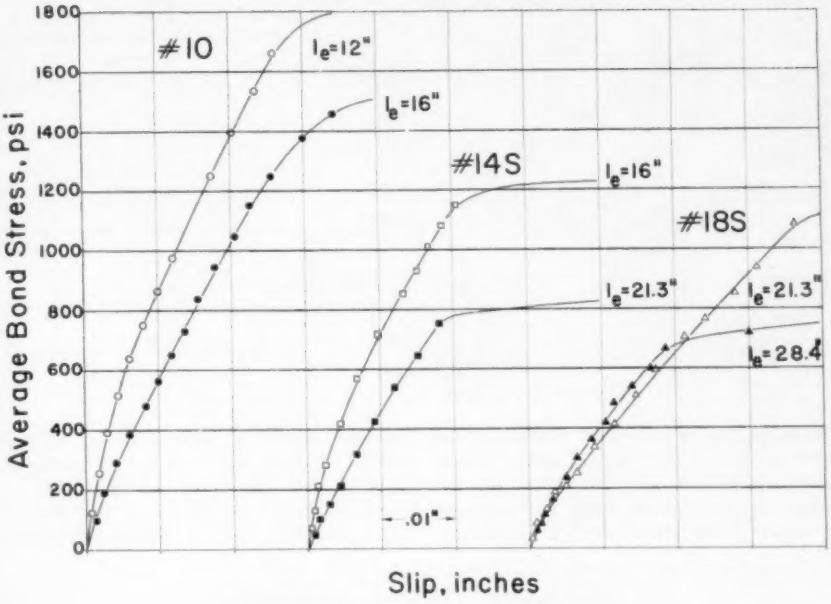

Fig. 2—Bond-slip curves, loaded end

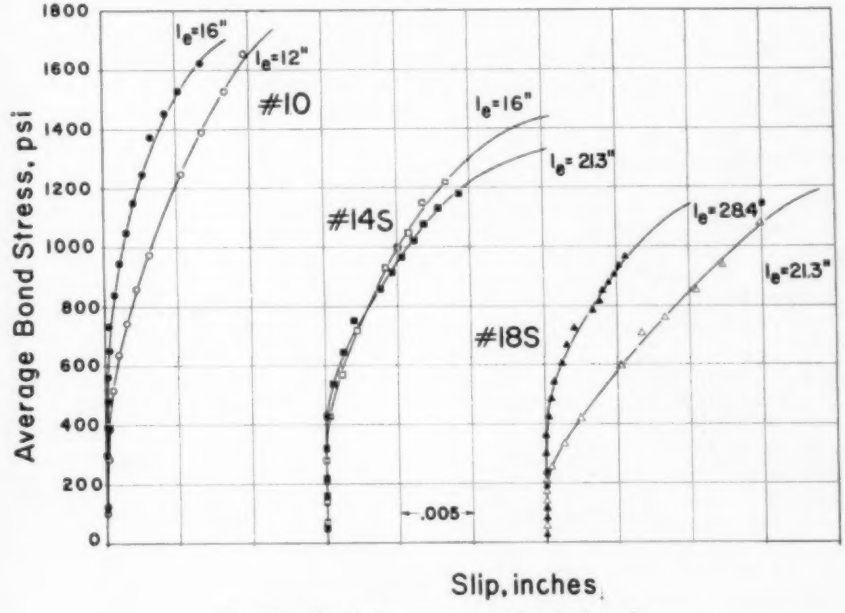

Fig. 3—Bond-slip curves, unloaded end

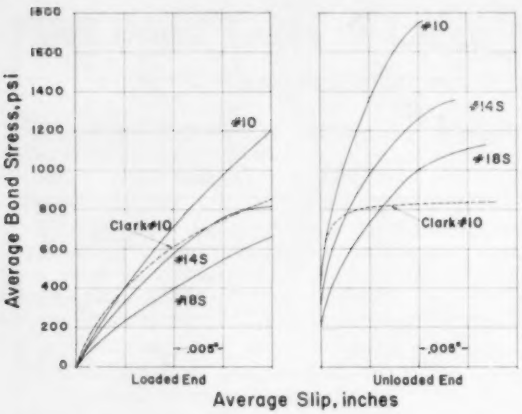

Fig. 4 — Bond-slip curves, average for two embedment lengths

facing surfaces of the prisms were cast against 2-in. steel plates, which were anchored into the concrete by short threaded rods. Load was applied by four R306 rams and a hydraulic system which is described in Reference 2. Slip was measured both at loaded and free ends of the bars at both prisms. Dial gages at the loaded ends were attached to a yoke consisting

Fig. 5—Bond-slip curves, corrected by scale factor

TABLE 1—BAR PROPERTIES

Deformed bar size No.	Bar weight, lb per ft	Nominal diameter, in.	Yield point,* psi	Bar deformations		
				Average height, in.	Average spacing, in.	Gap width, in.
10	4.202	1.270	77,000	0.064	0.57	0.130
14S	7.687	1.693	46,000	0.089	1.10	0.155
18S	13.190	2.257	38,000	0.103	1.55	0.210

*Determined in pull-out test at a slow loading rate as compared to that normally used in tension testing.

TABLE 2—PULL-OUT TESTS

Bar size, No.	Prism size, in.	Embedment length		7-day concrete strength, psi	7-day ultimate average bond stress, psi
		In.	Bar diameters		
10	9 x 9	12 16	9.4 12.6	3620 3500	1832 1724*
14S	12 x 12	16 21.3	9.4 12.6	3620 3900	1715* 1318*
18S	16 x 16	21.3 28.4	9.4 12.6	3000 4130	1192* 1165*

*Bar yielding before failure.

of a short steel tube and two welded brackets for the gage fastening. The tube was slipped over the bar and held in place by pointed set screws. Dial gages at the free end were mounted in the usual way. Strains in the steel were measured by SR-4 strain gages attached to the longitudinal bar gap midway between the two bearing plates.

RESULTS

Since the results of these "duplicate" tests agreed closely, the bond-slip curves of Fig. 2 and 3 are given as average values for the two prisms of each test. All specimens failed by splitting of the concrete, which in most cases was preceded by extensive yielding of the bar. The ultimate average bond stresses are listed in Table 2. Fig. 4 shows a comparison with Clark's bottom-cast #10 bar tests of the Hi-Bond bar. The average for both embedment lengths are given, since Clark's curves give such average values. Fig. 5 shows that, if geometric similarity is considered by reducing slip in proportion to bar and specimen size, the curves for various bar sizes tend to merge. Hence, the uncorrected slips at a certain bond stress are approximately proportional to bar diameter. For example, slip of the #14S bar is reduced in Fig. 5 by the ratio Diameter #10 bar/Diameter #14S bar.

APPLICATION IN DESIGN

The results of these pilot tests are presented here without design recommendations. It is hoped that the test data will be of aid to designers until more comprehensive information may become available at some future date.

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Anchorage Bearing Stresses in Post-Tensioned Concrete

By KARL H. MIDDENDORF*

The maximum allowable stress at post-tensioning anchorage in end blocks is based on an assumed formula, originally established by the Bureau of Public Roads in its criteria for prestressed concrete issued in 1954. These criteria have closely guided writers of specifications and codes for prestressing. Exception has been taken, it is assumed, only when subsequent study or experiments have indicated that liberalization of these standards were in the interest of good design.

At the time these standards were proposed the experiments of Komendant were given consideration. Recently this writer has become aware that some designers, apparently have concluded that Komendant's square root formula is more logical than the cube root formula adopted. For example, in one instance on a public works project, a 3⅝ in. diameter bearing having a net area of about 7.8 sq in. was centered 6 in. from the edge of the end block. With an approximate load of 90 kips the bearing plate stress approached 11,600 psi which divided by the square root of the A_c/A_p ratio (3.8) gives slightly over 3000 psi, which at 60 percent indicates an assumed f'_{ci} of 5000 psi.

In 1953, the writer, in developing a bearing anchor for a single 0.25 in. diameter headed wire found that an 0.625 in. diameter washer embedded in a standard 6 x 12 in. cylinder offered sufficient bearing to break the wire at 12,500 lb tension. Yet when the cylinder was carefully split open there was no indication, even under magnification, of excessive bearing pressure, though the bearing under the washer was almost 50,000 psi.

In 1957, the writer conducted the tests set out in Table 1. Assuming the relationship of 100 percent shown in Test 6, as the ideal relationship of plated cylinder to control cylinder, it would appear that the closer any formula used approached this relationship the more accurate it could be assumed to be. The writer contends that 98.4 percent is more logical than 85.4 percent.

When the recommendations of ACI-ASCE Committee 323 were issued, the cube root formula was retained but the percentage of f_c' raised 50 percent. It seemed strange that none of the designers who apparently have adopted the square root formula offered discussion on Section 207.3.4. Feeling that factual data was needed to establish the relationship between the two formulas, the writer decided on a series of tests.

These tests were set up to inquire into the following conditions of post-tensioning anchorage:

1. The constancy of the A_c/A_p relationship at various ages of the concrete from 1 to 28 days. Also to include a few tests on lightweight concrete to determine whether it followed regular concrete pattern. The results of these tests are shown in Table 2

2. The constancy of the A_c/A_p relationship under varying concentric patterns both for round and rectangular plates. Also the result of departure from geometrical similarity.

*Board Chairman, The Prescon Corporation, Corpus Christi, Tex.

TABLE 1—RELATIONSHIP OF VARYING SIZE ROUND PLATES CENTERED ON 8 x 16-IN. CYLINDERS*

Test No.	Plate diameter, in.	Plate area, sq in.	Failure load, kips	Cylinder diameter, in.	Cylinder area, sq in.	Ratio (A), $2\sqrt{\frac{A_c}{A_p}}$	Ratio (A) times plate load, kips	Percent ratio (A) load to cylinder, load, Test 8	Ratio (B), $3\sqrt{\frac{A_c}{A_p}}$	Ratio (B) times plate load, kips	Percent ratio (B) load to cylinder load, Test 8
1	3	7.07	79.5	8	50.26	2.66	211.6	106.0	1.93	154.1	77.1
2	4	12.56	98.4	8	50.26	2.02	198.7	99.6	1.59	155.9	77.9
3	5	19.63	121.3	8	50.26	1.59	193.4	96.9	1.37	166.2	83.2
4	6	28.27	144.3	8	50.26	1.33	192.3	96.3	1.21	174.8	87.3
5	7	38.48	159.9	8	50.26	1.14	183.1	91.7	1.09	173.9	87.1
6	8	50.26	199.6	8	50.26	1.00	199.6	100.0	1.00	199.6	100.0
	Average relationship to 100 percent							98.4			85.4

*All test samples made from same batch at same time under presumed uniform conditions. All cylinders tested by Raymond G. Osborne Laboratories, Inc., at 28 days.

TABLE 2—RELATIONSHIP OF UNIFORM 3⅛ IN. DIAMETER PLATES ON UNIFORM 6 x 12-IN. CYLINDERS*

Test No.	Age at test		Failure load plated cylinder, kips	Failure load control cylinder, kips	Ratio (A), $2\sqrt{\frac{A_c}{A_p}}$	Ratio (A) times plate load, kips	Percent ratio (A) load to cylinder load	Ratio (B), $3\sqrt{\frac{A_c}{A_p}}$	Ratio (B) times plate load, kips	Percent ratio (B) load to cylinder load
	Hr	Days								
				Regular concrete						
1	30	1†	4.0	10.0	1.92	7.7	76.8	1.54	6.2	61.5
2	83	3½†	16.0	25.0	1.92	30.7	122.9	1.54	24.6	98.5
3	97	4†	17.0	29.0	1.92	32.6	112.5	1.54	26.2	90.5
4	129	5†	21.0	32.0	1.92	40.3	126.0	1.54	32.3	101.1
5	172	7†	23.0	36.0	1.92	44.2	122.6	1.54	35.4	98.2
6	246	10†	22.0	42.0	1.92	42.2	100.5	1.54	33.9	94.7
7	358	15†	26.0	54.0	1.92	49.9	92.4	1.54	40.0	74.1
8		21	31.0	58.0	1.92	59.5	102.6	1.54	47.6	82.5
9		28	45.0	76.0		86.4	113.7	1.54	69.2	90.5
	Average relationship to 100 percent						108.9			87.9
				Lightweight concrete						
10	75	3†	34.0	80.0	1.92	65.3	81.6	1.54	52.5	65.5
11	225	9†	62.0	118.0	1.92	119.1	100.9	1.54	95.5	81.2
	Average relationship to 100 percent						91.3			73.3

*All cylinders as well as related control cylinders were cast from the same batch at the same time under presumed uniform conditions by T. Y. Lin and Associates in their laboratory. The design mix was for a 3000 psi concrete at 28 days. The tests were made to check bearing relationship at various times up to 28 days and also the relationship of regular concrete to lightweight concrete. Tests were made by T. Y. Lin and Associates in their laboratory.
†Approximate.

TABLE 3—RELATIONSHIP OF VARYING PLATE SIZES AND VARYING CORRELATED CYLINDERS*

Test No.	Age at test, days	Plate size, in.	Plate area, sq in.	Failure load of plated cylinder, kips	Cylinder size, in.	Cylinder area, sq in.	Failure load of control cylinder, kips	Ratio (A) $\sqrt[2]{\frac{A_c}{A_p}}$	Ratio (A) times plate load, kips	Percent, ratio (A), load to cylinder load	Ratio (B), $\sqrt[3]{\frac{A_c}{A_p}}$	Ratio (B) times plate load, kips	Percent, ratio (B) load to cylinder load
1	9	2 x 2	4	30.2	4 x 4	16	47.8	2.00	60.4	126.4	1.58	47.6	99.9
2	9	2 x 4	8	52.6	4 x 6	24	68.2	1.73	91.1	132.1	1.44	75.5	111.0
3	9	3 x 6	18	99.4	6 x 9	54	154.0	1.73	172.2	111.7	1.44	143.0	93.0
4	11	3 x 6	18	118.0	6 x 12	72	251.0	2.00	236.0	94.0	1.58	187.0	74.5
5	11	4 x 6	24	177.5	7 x 16	112	429.0	2.16	383.4	89.3	1.67	295.0	69.1
6	11	4 x 6	24	161.5	7 x 16	112	429.0	2.16	348.8	81.0	1.67	269.2	62.8
7	11	4 x 6	24	149.0	7 x 16	112	429.0	2.16	521.8	75.0	1.67	249.1	58.2
8	9	3 in. diam.	7.07	51.6	6 in. diam.	28.2	97.5	2.00	103.2	105.8	1.58	81.5	82.8
9	9	3 in. diam.	7.07	65.3	8 in. diam.	50.3	150.3	2.67	174.4	116.2	1.93	126.2	84.2
10	11	Control			6 in. diam.	28.2	105.0						
11	11	3 in. diam.	7.07	82.5	10 in. diam.	78.5	333.0	3.33	274.7	82.5	2.26	187.3	56.1
12	11	3 in. diam.	7.07	108.5	12 in. diam.	113.1	537.0	4.00	434.0	80.8	2.52	272.9	51.2
13	11	3 in. diam.	7.07	154.0	16 in. diam.	201.1	1120.0	5.33	820.8	73.3	3.06	470.3	42.3
Average relationship to 100 percent.										97.3			73.6
Average relationship to 100 percent eliminating tests 5, 6, 7.										102.5			88.3
Average relationship to 100 percent eliminating tests 5, 6, 7, and raising tests 11, 12, 13 — 20 percent.										107.7			91.7

*All test samples were made from the same batch at same time under presumed uniform conditions by T. Y. Lin and Associates in their laboratory. The design was for 5000 psi concrete in 28 days. Based on results of tests in Table 2, it was considered that accurate comparative tests could be made in 9 to 11 days. All cylinders were tested by T. Y. Lin and Associates except the control cylinders for tests 11, 12, and 13, which were made by Smith Emery Laboratories 14 days after casting.

The results of these tests are shown in Table 3

3. The constancy of the A_c/A_p relationship when the bearing plate was tilted in relation to 90 deg from the line of applied load. The results of these tests are shown in Table 4

4. The constancy of the A_c/A_p relationship with varying strength concretes. The results of these tests are shown in Table 5

DISCUSSION

1. The constancy of the A_c/A_p relationship in Table 2, considering possible variations in the mechanics of casting the different cylinders, seems fully established after the third day. The percentage rise of Tests 6, 7, and 8, indicate a faulty plate cylinder for Test 7. Here again the percentage relationship to 100 percent indicates that the square root formula is more nearly accurate than the cube root formula. The inadequate tests on lightweight concrete seem to indicate that constancy starts to exist some place between the third and ninth day.

2. The constancy of the A_c/A_p relationship in Table 3, seems fully established, where the areas are geometrically similar and concentric. Tests 4, 5, 6, and 7 show that when there is a departure from this pattern there is a reduction in the permissible stress. Computed however on the basis of a geometrical concentric block 7 x 9 in. the percentage of ratio load to control cylinder load would rise to 111 percent, indicating that additional width in one direction may add some factor of safety. Tests 5, 6, and 7, were similar except that in Test 6, a closed welded loop of #3 bar was placed 3 in. in front of the bearing plate, while in Test 7, two 0.25 in. diameter headed wires were placed 3 in. in front of the bearing plate. It was thought this reinforcement would strengthen the block. Apparently it either interfered with placement enough to reduce the bearing under the plate or may have aided fracture by reducing the shear area in the concrete. The relationship

TABLE 4—RELATIONSHIP OF 4 x 4 IN. PLATE CENTERED ON 7 x 7 x 12 IN. BLOCK EXCEPT FOR TEST NO. 1, WHERE A 3 IN. DIAMETER PLATE WAS USED*

Test No.	Tilt from 90 deg	Failure load plate block, kips	Failure load control block, kips	Ratio (A), $2\sqrt{\frac{A_c}{A_p}}$	Ratio (A) times plate load, kips	Percent, ratio (A) load to control	Ratio (B), $3\sqrt{\frac{A_c}{A_p}}$	Ratio (B) times plate load, kips	Percent, ratio (B) load to control
1	none	91.6	133.6	2.63	240.9	180.3	1.91	173.0	130.0
2	none	83.8	133.6	1.75	146.6	109.7	1.45	90.1	90.1
3	1 to 32 1 deg - 47 min	80.9	133.6	1.75	141.6	105.9	1.45	88.2	87.9
4	2 to 32 3 deg - 34 min	88.6	133.6	1.75	155.0	116.0	1.45	95.9	95.9
5	3 to 32 5 deg - 21 min	89.6	133.6	1.75	156.8	117.3	1.45	97.8	98.1
6	4 to 32 7 deg - 8 min	95.5	133.6	1.75	167.1	125.0	1.45	104.2	103.8
Average relationship to 100 percent						125.7			100.9
Average relationship to 100 percent eliminating Test 1						114.8			95.2

*The test was conducted to determine the effect of tilting the plate at varying angles from 90 deg to the center line of loading. Test blocks were cast by T. Y. Lin and Associates, from the same batch at the same time and under presumed uniform conditions. The design mix was 5000 psi at 28 days. Based on the results of tests in Table 2, it was considered that accurate comparative tests could be made in 9 days. All tests were made by T. Y. Lin and Associates in their laboratory.

TABLE 5—RELATIONSHIP OF UNIFORM 3 IN. DIAMETER PLATES ON 6 x 12 IN. CYLINDERS WITH VARYING DESIGN MIXES

Test No.	Design mix nominal	Failure load plates cylinder, kips	Failure load control cylinder, kips	Ratio (A), $2\sqrt{\frac{A_c}{A_p}}$	Ratio (A) times plate load, kips	Percent, ratio (A) load to control load	Ratio (B), $3\sqrt{\frac{A_c}{A_p}}$	Ratio (B) times plate load, kips	Percent, ratio (B) load to control load
1	2000	25.8	58.3	2	51.6	88.5	1.58	41.0	70.0
2	3000	38.9	81.6	2	79.8	97.8	1.58	61.5	75.5
3	4000	43.3	102.5	2	86.6	84.5	1.58	68.5	67.0
4	4500	55.3	119.3	2	110.5	92.7	1.58	87.2	73.8
5	5000	43.3	145.0	2	86.6	59.7	1.58	68.2	47.1
6	6000	63.3	162.3	2	126.5	77.9	1.58	99.0	61.2
Average relationship to 100 percent						83.5			65.8
Average relationship to 100 percent eliminating Test 5						88.3			69.5

Test samples were cast by Raymond G. Osborne Laboratories and tested by them at 28 days.

in Tests 11, 12, and 13, is not an accurate reflection of the constancy of the A_c/A_p relationship since, the control cylinders could not be broken on the same day as the plate cylinders and probably gained 20 to 25 percent strength in the intervening time. When this is considered, the relationship still is constant. These tests also indicate the square root formula is more reliable than the cube root formula. Tests 9, 11, 12, and 13, indicate that the restriction to a maximum bearing stress of f_c' is not realistic.

3. The constancy of the A_c/A_p relationship does not seem to be affected by reasonably small tilting of the bearing from 90 deg with the line of applied force. Test 1, also indicates that the f_c' maximum is not realistic. This series of tests possibly should be extended to determine the maximum tilt that installation supervision should permit.

4. Test 5, shows something was wrong with the plated cylinder since it failed in a lesser load than the 4500 psi concrete and the same load as the 4000 psi concrete. The other tests indicate the constancy of the A_c/A_p relationship throughout the normal range of concretes used in structures. It would be interesting to see if the pattern persists in the higher strength concretes.

CONCLUSION

The test results strongly indicate that reconsideration be given to the recommendations in Section 207.3.4 of the Committee 323 recommendations to change $3\sqrt{A_c/A_b}$ to $2\sqrt{A_c/A_b}$. Further, that the test results indicate that the allowable value of f_{cp} should be increased to not exceed twice the value of f'_{ci}.

CONTRIBUTIONS TO THE ACI JOURNAL

Technical pages of the ACI JOURNAL offer a medium for reporting what is new and interesting in concrete research, design, construction, maintenance, or manufacturing. ACI members and JOURNAL readers are urged to share their knowledge and experience by submitting papers and reports for publication in the ACI JOURNAL.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Repairs on the bridges of the Gafsa railroad, on the Seldja Wadi, Tunis (Les travaux de reparation des ponts du chemin de fer de Gafsa, sur l'oued Seldja, Tunisie)

HENRY LOSSIER, *Le Génie Civil* (Paris), V. 136, No. 12, June 15, 1959, pp. 264-267
Reviewed by ARON L. MIRSKY

The Sfax-Gafsa railroad crosses and recrosses the Seldja Wadi, which is subject to sudden and disastrous flash floods. Article describes permanent repairs to the substructures of three of the bridges, which have been damaged and repaired many times since their construction.

Problem of bridge deck deterioration

H. B. BRITTON, *Roads and Streets*, V. 103, No. 5, May 1960, pp. 92, 95-96

Emphasizes that research for new answers and tighter construction control are needed to build more durable decks. The author poses some questions which might, through experience gained or research required, ultimately eliminate failures of structural decks and wearing courses. Covering the areas of design, specifications, construction, inspection, admixtures, concrete quality, and administrative control, the comments and questions could also serve as a partial check list of what should be done in designing and constructing bridge decks.

Load distribution on a prestressed concrete slab bridge

A. C. SCORDELIS, W. SAMARZICH, and D. PRITZ, *Journal*, Prestressed Concrete Institute, V. 5, No. 2, June 1960, pp. 18-33

Analytical and experimental studies were made for the transverse distribution of wheel loads on a full scale simply supported prestressed concrete bridge. The slab is post-tensioned in two directions. Published results using the classical elastic theory were applied to obtain influence surfaces for longitudinal moments in the slab. Experimental values for longitudinal moments produced by a truck loading on the bridge were obtained, and these are compared with theoretical values. The effect of transverse prestressing on load distribution was also measured experimentally.

Authors concluded that: 1. The load distribution on this simple span prestressed concrete slab bridge, as indicated by the actual bending moments, agrees fairly well with the elastic theory. Experimental values are generally lower than theoretical values with the greatest discrepancy occurring at points near the wheel loads; 2. Transverse prestressing had no measurable effect on the transverse and longitudinal moments produced by wheel loads; 3. Reliable experimental values can be obtained by the use of Carlson gages and with oscillograph recordings, even when the strain readings are relatively small; 4. Since the maximum live stress produced by the heaviest

A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 5, Nov. 1960, *Proceedings* V. 57. Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are *not* available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

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obtainable experimental truck was no more than 200 psi, while the slab was designed to carry a live load fiber stress of 1320 psi, it was evident that the present empirical rule for the design of slabs of long spans and large width is too conservative from the point of view of stresses. Even the simultaneous loading of all lanes with full impact is not likely to result in stresses as high as those assumed in design. Consequently, a more rational approach to the design would result in considerable saving in prestressing steel, and in a much better behavior of the slab, with less camber and creep.

Additional thoughts on the waterway areas of small highway bridges (Nouvelles considerations sur les debouches des petits ouvrages sous routes)

A. DELORME, *Annales des Ponts et Chaussées* (Paris), V. 129, No. 6, Nov.-Dec. 1959, pp. 681-713

Reviewed by ARON L. MIRSKY

Following up previous paper on determination of waterway areas [*Annales*, Mar.-Apr. 1959, pp. 141-167; Current Reviews, ACI JOURNAL, V. 30, No. 9, Mar. 1960 (*Proceedings*, V. 56) p. 891] author discusses culvert hydraulics and develops a formula for discharge capacity in terms of upstream head, slope, and downstream conditions. The concept of critical slope is emphasized. Numerical examples are given, and experimental verification investigations are described. Author concludes with series of practical suggestions.

Reinforced concrete railroad viaduct, Papagaio Valley, Brazil (Le viaduc ferroviaire en beton arme de la vallee du Papagaio, Bresil)

Le Génie Civil (Paris), V. 136, No. 21, Nov. 1, 1959, p. 447

Reviewed by ARON L. MIRSKY

Brief description of an interesting structure over the Papagaio River, State of Parana, which consists of eight parabolic arches of 34 m average height over a valley 600 m wide and 40 m deep. The arches (seven of 42.5-m span, one of 90 m) are of open-spandrel type, with the deck supported on slender columns. Structure is founded on rock, but the tubular steel centering used during construction was supported on temporary timber piles.

A rigidframe bridge with three-dimensional curvature in Caracas (Raumlich gekrummte Rahmenbrucke in Caracas)

J. JOHANNSON, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 1, Jan. 1960, pp. 1-6

Reviewed by RUDOLPH SZILARD

Construction of a new shopping center on the top of a conical hill in Caracas, Venezuela, has required for architectural reasons, a highway in screw tread form. For the overpass at Avenida de Fuerzas a bridge was designed having a space curvature. Preliminary investigations have indicated that a prestressed rigid frame bridge with hollow box section is the most suitable to resist the torsional moments. The bridge is six times statically indeterminate. The redundant forces, moments, etc., were determined by means of the "classical" methods. Author describes the construction methods including prestressing.

Bridges: Bearing pads, foundations, scour and waterways

Bulletin No. 242, Highway Research Board, Mar. 1960, 77 pp., $1.40

Contains six research papers presented at the 28th annual meeting of the Highway Research Board:

"Elastomeric Bridge Bearings," by R. L. Pare and E. P. Keiner, and "Report on Tests of Neoprene Pads Under Repeated Shear Loads," by A. M. Ozell and J. F. Diniz, describe research on elastomeric (rubber-like) materials which show considerable promise of being particularly suitable for seating precast bridge girders.

"Tests of Steel Tubular Piles Driven Near Saigon River, Vietnam," by D. Allan Firmage, and "A Rational Method for Determining Safe Foundation Pressures and Embankment Stability," by S. Lifsitz, present two rather widely

different engineering approaches to the problem of safe foundation design, possibly accounted for by differences in geology of the construction locations.

Bridge girders prestressed with large strands

Concrete and Constructional Engineering (London), V. 55 No. 6, June 1960, pp. 233-238

Construction details of bridge girders which support a two-story office building built over a canal. The 79 ft span girders were precast in eight sections stressed with 1⅛-in. diameter steel strands.

Catalog of highway bridge plans

U.S. Bureau of Public Roads, U.S. Government Printing Office, Washington 25, D.C., 1959, 167 pp., $1

Catalog contains available bridge designs dating as far back as 1950 for H20-S16 loading or heavier, received in reply to letters to state highway departments. Catalog, which is divided into 13 sections, contains over 4500 designs in all. Designs have been listed by states according to span types, simple, continuous, or cantilever, in their relevant sections and cover-rolled beam spans with and without composite action, steel deck girder spans riveted or welded, with and without composite action, steel truss spans, reinforced concrete slab, deck girder, box girder, and rigid frame spans, prestressed concrete slab, girder and box girder spans, steel arch spans, reinforced concrete arch spans, movable bridges of lift span and bascule span types, suspension bridges, pedestrian overcrossing structures, traffic sign structures, and tunnels.

The last section contains quantity charts which may prove useful in making design comparisons. These charts were prepared from data tabulated in the master catalog and show structural steel quantities for rolled beam, deck plate girder and truss spans, and concrete quantities for reinforced concrete spans. Insufficient data were available to prepare charts covering reinforcing steel.

The master catalog was prepared primarily for use by state highway departments to facilitate the exchange of designs on a national basis and all requests for design plans listed should be made directly to the state highway departments listing them.

Construction

Unique features at Blakehurst high school

Constructional Review (Sydney), V. 33, No. 3, Mar. 1960, pp. 18-21

Auditorium roof is a concrete folded plate 3½ in. thick spanning 60 ft. The main classroom has an 8-in. plate floor. Design considerations, construction features, and costs are discussed briefly.

Shale scuttles reservoir design

Engineering News-Record, V. 165, No. 4, July 28, 1960, pp. 41-42

The floor of this concrete reservoir had to be redesigned after construction had already begun because a shale formation exposed to heavy rains during excavation has swelled as much as 4 in. The solution was a floor system of thin shell concrete barrel arches on drilled, cast-in-place caissons. The 20-ft chord barrel shells are on caissons at 5 ft on centers. The shells have a rise of 1 ft 10 in. in 10 ft and clear the ground a minimum of 18 in. When completed the floor system will carry 24 ft of water.

Construction of steam power plants in Hungary (Bau von Dampfkraftwerken in Ungarn)

Laszlo Mokk, *Bauplanung-Bautechnik* (Berlin), V. 13, No. 8, Aug. 1959, pp. 347-352
Reviewed by J. F. Leppman

Describes three power houses constructed of precast concrete. Two were designed after the pattern of steel frame structures, the first with frame elements of solid web I-sections, the second with Vierendeel type columns and truss type roof with crane girders. The third employed a new design concept, the floor being formed of free spanning thin walled tied arch seg-

ments while the walls were folded plates. The ratio of volume of concrete to enclosed space was thus reduced from 0.014 in the first of these structures to 0.0065 in the last, reinforcing steel from 0.124 to 0.079 lb per cu ft of enclosed space. All concrete elements were cast at the site; the longest (120 ft) and heaviest (66 tons) were cast directly on the power house floor to avoid horizontal hauling. Most connections were made by welding projecting ends of reinforcing steel.

Concrete architecture in Scandinavia

Engineering (London), V. 188, No. 4880, Oct. 30, 1959, p. 410

Reviewed by Aron L. Mirsky

Brief report of current construction in concrete in Scandinavian countries, as noted on a visit by a group of architects and engineers in September. Particularly noted are the trend to prefabrication and on-site mechanization, and the use of lightweight concrete for insulation and interior partitions and of exposed-aggregate concrete.

Reinforced concrete jetty for the Canada-India reactor project

T. S. Raman, *Indian Concrete Journal* (Bombay), V. 34, No. 4, April 1960, pp. 127-132

A reinforced concrete piled jetty carries the sea water pipe lines of the Canada-India Reactor at the Atomic Energy Establishment, Trombay, for 3200 ft and ends up with a jetty head and caisson. The jetty is built of several precast units but the deck which carries the intake pipe line as well as the roadway is cast in situ. The construction of the jetty involved the manufacture of quality concrete for use in the various members, and it is with this particular aspect which includes proportioning of the mix, the various control measures adopted in the field, and the statistical analysis of the crushing strengths of concrete test cylinders that this article is concerned.

Approximately 200,000 ft of concrete involving about 2400 tons of cement and 750 tons of steel have been placed in the construction. It would have been possible neither to make the best utilization of these materials nor to produce concrete of a consistently high quality except through the enforcement of quality control measures. The grading of aggregates to make a cohesive concrete aided by vibration to make it dense and impervious are essential to prevent the deterioration of a marine structure. All said and done the production of the controlled concrete for the construction of the jetty at the Canada-India reactor was a success particularly when it was found that economy in materials could be attained.

Underground station for Western District Post Office, London

Proceedings, Institution of Civil Engineers (London), V. 15 Session 1959-60, Feb. 1960, pp. 81-104

Design and Control

Stuart Blackford and Eric W. Cuthbert, pp. 81-94

Construction

Vincent H. Collingridge and Richard E. Tuckwell, pp. 95-104

Reviewed by Aron L. Mirsky

Station, a three-story structure 180 x 90 ft in plan, is entirely underground, the base being 70 and the "roof" 32 ft below the surface. Roof forms floor of the basement post office garage. Construction was in open cut. End walls of structure, coincident with the site boundaries, extend up to ground surface; they are pierced by 12- and 9-ft diameter cast iron-lined tunnels for the underground railway, and are prestressed.

First paper, besides describing project, gives details of two- and three-dimensional photoelastic investigations and vibrating-wire strain gage tests designed to verify design assumptions, and of tests on the loss of prestress from shrinkage and creep and friction between cables and ducts. Second paper includes brief discussion of the relative merits of gap- and continuous-graded concrete (the contractor adopted the former for concrete without prestress-

ing cables — the latter for concrete containing cables) and of methods of handling large numbers of prestressing cables on site.

Construction Techniques

Ocrating process offers acid-resistant concrete

Concrete Products, V. 63, No. 7, July 1960, pp. 46-48

Brief description of silicon tetrafluoride gas treatment of concrete products. The gas reacts with the concrete (in a special sealed chamber) to alter chemical structure with the aim of increasing its resistance to acids and wear.

Construction with prefabricated elements, critics of design and construction with the help of time studies (Bauen mit Stahlbetonfertigteilen, Kritik von Konstruktion und Ausfuhrung mit Hilfe von Zeitstudien)

RUDOLF SEELAND, *Beton und Stahbetonbau* (Berlin), V. 55, No. 2, Feb. 1960, pp 31-35
Reviewed by RUDOLPH SZILARD

Prefabricated structures should be compared not merely on the basis of material savings, workmanship and good performance, but also the time element required for erection should be considered. Author compares prefabricated structures built in Hungary, Finland, and Germany. These structures are either entirely prefabricated or put together using prefabricated elements fastened to each other by prestressing.

Hidden profits in truck mixer operation

A. C. THOMAS, *Modern Concrete*, V. 24, No. 2, June 1960, pp. 48-50

There are many factors involved in the total profit picture of a ready-mixed concrete business, but article restricts discussion to a few which specifically concern the operation of truck mixers. Investigates some of the practical points of truck mixer operation, which, if performed efficiently, can measurably increase profits. Briefly stated, these are: (1) achieve the fastest possible charging time of mixer drum under batching plant; (2) produce thoroughly mixed, uniform concrete in transit to permit immediate discharge as soon as the mixer is spotted, or if mixing must be done at the job site; (3) achieve the quickest possible discharge time; (4) minimize down time due to mechanical failures; (5) last and most important — choose the correct size and type of mixer for the particular operation.

Note

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Handbook of heavy construction

FRANK W. STUBBS, JR., (Editor) McGraw-Hill Book Co., New York, 1959, 1040 pp., $18.50

This handbook is a reference book of practical information and data prepared for those engaged in heavy construction. The book, written for the constructors or general practitioners, will assist in reviewing their own specialty but also by presenting important aspects of related operations and procedures. Thirty-six noted specialists supply expert guidance in estimating the requirements of a construction job. Sections cover excavation and transportation of earth and rock, concrete, steel erection, timber construction, highways, pipelines, and foundations. In addition miscellaneous equipment and operations are discussed with special attention given to costs, materials, procedures, equipment, controls, schedules, and contractors operations. Tables are provided for figuring the cost of operating and owning equipment, out-

put of equipment, economical use of materials, and other cost factors.

The designer, consulting engineer, architect, or owner, may profit from this handbook by gaining a better understanding of the construction problems involved in a project and thereby simplify the structure specifications. This can also be a valuable reference for students and professors when they approach the over-all aspects of a project.

Dams

Kariba hydroelectric scheme on the Zambezie, Rhodesia (L'amenagement hydroelectrique de Kariba, sur le Zambeze, Rhodesie)

Le Génie Civil (Paris), V. 136, No. 22, Nov. 15, 1959, pp. 453-460
Reviewed by ARON L. MIRSKY

An interesting, well-illustrated description of the Kariba project, the power plant, the thin doubly-curved arch dam, and of the construction procedures.

Inspection of reinforcement in large structures with special reference to the Bharkra project

N. K. BERRY, *Indian Concrete Journal* (Bombay), V. 34, No. 4, Apr. 1960, pp. 133-136

Among the things inspected were quality of steel, cleanliness of bars, bending of bars, spacing of bars, splices, welding, embedment of reinforcement. A brief description of the layout of the fabrication yard at Bharkra Dam is also presented.

Investigation of a simple criterion characterizing the "cantilever" effect in arch dams (Recherche d'un critere simple caracterisant l'effet "console" dans les barrages-voutes)

J. ESTIENNE, *Le Génie Civil* (Paris), V. 136, No. 22 Nov. 15, 1959, pp. 461-465
Reviewed by ARON L. MIRSKY

Starting from basic principles and making certain simplifying assumptions (e.g. constant radius), author develops expressions for the deflection of the vertical cantilever slice at any height and for the moments, for both fixed and hinged arches. The influence of the foundation and the effect of temperature changes are discussed, and the method is applied to certain dams in southwestern France.

Statical investigations of the draft-tube floors in hydropower plants (Statische Untersuchungen uber die Saugschlauchdecken in Wasserkraftanlagen)

RUDOLF KUHN, *Der Bauingenieur* (Berlin), V. 34, No. 5, May 1959, pp. 192-200
Reviewed by ARON L. MIRSKY

Theoretical analysis based on plate theory having proved ineffective, resort was had to photoelastic investigations and measurements on the floor of Turbine No. 4 of the Jochenstein Dam on the Donau. Author suggests an improved method of analysis, and shows how his proposed method of dividing slab into strips on elastic supports gives results approximating action of floor as shown by photoelastic results.

Design

Ultimate strength design of reinforced concrete cross sections

G. FEKETE, *Journal*, Institution of Engineers, Australia (Sydney), V. 32, No. 6, June 1960, pp. 133-135

A design graph for the ultimate strength design of rectangular reinforced concrete cross sections is developed and its use is shown.

Doubly reinforced concrete beams proportioned by permissible stresses under CA2-1958

PAUL B. JONES and T. JUMIKIS, *Constructional Review* (Sydney), V. 33, No. 4, Apr. 1960, pp. 21-29

Presents a method for rapid calculation of doubly reinforced concrete beams based on the permissible stresses of the code for concrete in buildings CA2-1958, as established by the Standards Association of Australia. This code is similar to the ACI Building Code. Examples are given for various applications and tables are given to make the calculation of steel simple and direct. The semielastic method is presented to overcome difficulties arising

from the use of a constant (n) with some discussion on the relationship between permissible stress and ultimate strength bases as is applicable to this problem.

Analysis of symmetrical single-bay multiple-story frames

K. Brzezinski, *Concrete and Constructional Engineering* (London), V. 55, No. 4, Apr. 1960, pp. 147-153

Analysis by moment distribution method, using two operations only, for any arrangement of loads, of a single-bay, multistory frame. Moments are computed for only half the frame, usually the part to the left of the axis of symmetry.

Degree of fixity between a column and foundation (Uber den Einspanngrad einer Stutze im Fundament)

K. Oplanden, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 2, Feb. 1960, pp 35-44
Reviewed by Rudolph Szilard

In general rigid frames are analyzed using total fixity or hinged column end conditions. The actual condition is between the two extremes. The author shows how to determine the degree of fixity, which is a function of the soil condition, and the shape and size of the foundation. The use of partial fixity for different computation methods including moment distribution is treated extensively. Numerical examples show the actual application.

Ultimate strength design of concrete sections eccentrically loaded (in Italian)

G. Giannelli, *Giornale del Genio Civil*· (Rome), V. 96, No. 12, Dec. 1958, pp. 828-836
Applied Mechanics Reviews V. 13, No. 6, June 1960

Paper is devoted to the discussion of the proposed ultimate design specifications to be introduced in Italian design practice. Attention is focused on the ultimate strength of rectangular members subjected to combined bending and axial load. Because no particular ultimate strength equations are recommended, author derives formulas concerning the mentioned case. The following basic assumptions are introduced: (1) the diagram of compressive concrete stress is assumed to be a trapezoid; (2) plane sections normal to the axis remain plane after bending; (3) tensile strength in concrete is neglected; (4) maximum fiber stress is assumed to be 80 percent of the cube strength; (5) stress in tensile and compressive reinforcement is assumed not to exceed the yield point; (6) maximum steel strain and maximum concrete strain in flexure are assumed to be known.

Using the equilibrium equations, a set of relationships is derived. The following particular cases are considered: (a) concrete failure, steel stress below yield point; (b) concrete failure, steel stress at the yield point; (c) steel failure; (d) balanced design. The relationships derived in the case of balanced design are plotted in a graph.

Design of slabs by the yield-line method

A. J. Ashdown, *Concrete and Constructional Engineering* (London), V. 55: No. 6, June 1960, pp. 221-226; No. 7, July 1960, pp. 271-278

A two-part article on the application of Johansen's method to the design of slabs supporting uniformly distributed load, two concentrated loads symmetrically disposed, load concentrated and strip parallel to span, rectangular slabs under various conditions, and circular and triangular slabs.

Analysis of rigid frames with legs fixed subterraneously (Berechnung von Rahmen mit im Untergrund eingespannten Stielen)

Gustav Jenne, *Der Bauingenieur* (Berlin), V. 34, No. 9, Sept. 1959, pp. 349-353
Reviewed by Aron L. Mirsky

Fixing bottoms of legs of dolphins by means of long piles results in increased ability to absorb lateral loads and reduced deflections. Presented here are expressions and both a chart and a nomograph for the analysis of such frames, including the determination of depth of embedment.

Nomograms for the design of beams and slabs by the load-factor method

J. C. STEEDMAN, *Concrete and Constructional Engineering* (London), V. 55: No. 2 Feb. 1960, pp. 91-100; No. 3, Mar. 1960, pp. 133-138; No. 4, Apr. 1960, pp. 169-174

Nomograms are presented for design of concrete members according to the load-factor method recommended in British Standard Code No. 114 (1957). Part 1 covers rectangular beams and solid slabs with reinforcement in tension only; Part 2, rectangular beams with reinforcement in tension and compression; and the final installment deals with rectangular beams and solid slabs with reinforcement in tension only and flanged beams with reinforcement in tension and compression.

Application of the relaxation method to the computation of shells

M. MIHAILESCU and Y. MIHAILESCU, *Indian Concrete Journal* (Bombay), V. 34, No. 4, Apr. 1960, pp. 147-151

AUTHORS' SUMMARY

The problems of elasticity, like most problems of applied mathematics, are characterized by the difficulties which arise when deriving solutions which satisfy the real boundary conditions, the existence and the uniqueness of which must be implicitly proved.

An advantageous procedure for integrating partial derivative equations consists of transforming into an algebraic system of finite difference relations obtained with a sufficient degree of accuracy from 100 to 400 equations, for the solution of which the relaxation method elaborated by Southwell seems to be appropriate.

The aim of this paper is to analyze the possibilities of applying the relaxation method to the computation of shells, a matter which was previously discussed in a more restricted manner by Flügge, Jenkins, and Arup.

The relaxation method is particularly suitable for elliptic shells for which the computation by the membrane hypothesis is sufficient, whatever be the loading or the support conditions on the boundary equivalent to the loading.

For these shells, which in statics resemble spatial latticed girders, moments and shearing forces are secondary stresses which occur due to the deformation of the element, and may be avoided through constructive measures.

Remarks on the design of silos (Remarques sur le calcul des silos)

HENRI PAMELARD, *Le Génie Civil* (Paris), V. 136, No. 23, Dec. 1, 1959 pp. 490-492

Reviewed by ARON L. MIRSKY

A study of the pressures and forces acting on the walls and hoppers of silos, during periods of filling, of static storage, and of emptying. Author indicates that the appropriate formulas usually employed give unsafe results, and cites provisions of the German standards (exact references to DIN numbers are unfortunately not given). For the period of filling, he derives an expression, based on an assumed parabolic pressure distribution, which is simpler than Janssen's and more conservative than either Janssen's or Reimbert's.

On the accounting for crack formation in the statical treatment of masonry structures

KYOSTI ANGERVO, *RILEM Bulletin* (Paris), No. 3, New Series, July 1959, pp. 71-82

AUTHOR'S SUMMARY

In building practice it is quite often necessary to employ frame structures with legs of masonry or unreinforced concrete. The statical treatment of such frame structure can be performed if the angular displacements, in the various loading cases, of a pillar of one-story height without tensile strength are known.

An investigation relating to this subject has been carried out in the Laboratory of Statics and Bridge Building of the State Institute for Technical Research, Helsinki, Finland. It complements the theory of a pillar without tensile strength under eccentric load to include the case that the forces act with eccentricities of different magnitude on both ends of the pillar. This

consideration has been performed below the proportional limit and it also accounts for the possible case that the curvature of the elastic line changes its sign at some point in the pillar. Formulas have been derived for the angular displacements at the end points of the pillar and the most important among them have made into nomographs. The angular displacements due to lateral displacement of the joints of a displaceable structure have also been taken into account in them.

For the treatment of frame structures a suitable iterative calculating method is presented, as well as an example of calculation concerned with a structure in practice. It is followed by a discussion in which the results obtained in this manner are compared to those obtained according to conventional methods, presupposing the existence of tensile strength. In the presented case considerable differences occur only in the upper stories of the building.

Beam factors for flexural members including beams with variable moment of inertia and prestressing (Belastungslieder fur Biegestabe mit Einschluss von Balken mit veranderlichen Tragheitsmoment und Vorgespannten Staben)

J. KAMMENHUBEN and H. WEGMANN, *Beton und Stahlbetonbau* (Berlin), V. 55, No. 1, Jan. 1960, pp. 7-22

Reviewed by RUDOLPH SZILARD

The purpose of the article is to extend the usefulness of various methods used in the solution of highly indeterminate frame structures. In general the fixed-end moments and the tangential angle of the deflection line are required in the solutions using moment distribution, slope deflection, and three moment equations.

Authors have introduced a couple of new definitions and have determined various influence lines for different conditions. The variation of moment of inertia as well as prestressing are considered. Moment coefficients are derived.

WRI building design handbook

Wire Reinforcement Institute, Washington, D.C., 1960, 168 pp., $3

This handbook for the use of welded wire fabric by designers, engineers, and architects, is divided into two parts. The first section contains the story of welded wire fabric, its overall reinforcing advantages, sections of the ACI Building Code applicable to fabric, short span construction, typical applications, research, and detailed information about important projects.

The second section is for the concrete designer and contains design tables with recommended welded wire fabric styles for one-way solid concrete slabs, flat slabs, tilt-up concrete wall panels, basement walls, retaining walls, and square individual column footings.

Geometrical criteria for the calculation of shells

A. BELES and M. MIHAILESCU, *Indian Concrete Journal* (Bombay), V. 34, No. 4, Apr. 1960, pp. 143-146

AUTHORS' SUMMARY

It is possible to devise the most varied types of roof surfaces, meeting both functional and aesthetic requirements, to cover large areas, but their application in practice is often prevented by mathematical difficulties because there is no known method of calculating shells of any shape and any boundary conditions. In this paper the authors have endeavored, by using the theory of characteristics of equations with partial derivatives, to develop a series of geometric criteria which would enable approximate calculations to be made for such shells.

It may be pointed out that the construction of complete edges in accordance with the membrane theory for the hyperbolic and parabolic shells may be, in many cases, unsatisfactory from a structural and functional point of view — a difficulty which seldom appears in shells of elliptical shape.

When the bearing does not satisfy in certain sections the conditions corresponding to the membrane theory,

the bending theory calculus offers larger possibilities of securing the equilibrium of the shell, because it appears as an integral of a differential of a higher degree.

Northlight shells for industrial buildings (Shedhallen fur Industriehallen)

W. SCHAEFFLER, *Beton-Herstellung und Verwendung* (Düsseldorf), V. 10, No. 3, Mar. 1960, pp. 107-111

Reviewed by FERDINAND S. ROSTASY

Northlight shells as a widely used roof construction for industrial buildings render a series of important advantages such as: uniform daylight illumination, aesthetic appearance, low maintenance costs, and economy of construction. The author discusses the characteristic features of this construction and outlines the statical analysis briefly as well as the range of application of this shell type. To obtain economic solutions it is important to design the falsework for fast errection and repeated use. Examples are given. The requirements with respect to the arrangement of the reinforcement and concrete are stated.

Materials

Effects of copper, lead, and zinc compounds and sugar on the setting of portland cement and plaster of Paris (in Japanese)

MANJIRO NAKAHARA, JUNICHI KASAI, TSUGUO MIYAKAWA, SHIGEKI AOKI, YOSHIKAZU KUNITOMO, and TOSHIDO KADOKURA, *Sekko to Sekkai*, No. 41, 1959, pp. 25-32

CERAMIC ABSTRACTS
Feb. 1960

The addition of compounds insoluble in H_2O, such as $Cu(OH)_2$, Cu_2O, $3CuO \cdot CuCl_2 \cdot 3H_2O$, $3CuO \cdot CuSO_4 \cdot 3H_2O$, ZnO, and PbO, and of sugar, retards the setting of portland cement but has no effect on the setting of plaster of Paris. When these compounds are added, the setting of plaster of Paris is retarded in the presence of $Ca(OH)_2$ but not always in the presence of NaOH. These results indicate that the retarding action of these compounds is due mainly to the formation of complex salts between the compounds and $Ca(OH)_2$.

Will it hold?

Building Construction Illustrated, V. 30, No. 2, Aug. 1960, pp. 44-46

This article briefly discusses the structural adhesives which are rapidly coming into use as new compounds and applications are developed. The article discusses the forms of adhesives, some of the research presently being carried on, and some typical applications of various types of compounds or proprietary products.

Effects of calcium chloride on portland cement

Allied Chemical Corp., Solvay Process Div., New York, 1958, 38 pp.

BRI ABSTRACTS
June 1959

This booklet is designed to outline the effects of calcium chloride on portland cement, concrete and mortars as obtained from laboratory and field tests conducted by recognized authorities. It contains tables, graphs and charts taken or drawn from the various reports and a bibliography with abstracts. A final section presents 24 questions and answers about the use of calcium chloride with portland cement.

Air jig saves beneficiating dollars

KNEELAND A. GODFREY, JR., *Rock Products*, V. 62, No. 10, Oct. 1959, pp. 106-108, 110

To meet the specifications of the state highway department for aggregate, this producer installed a 150-tph Baum air operated jig. This type of separatory device is brand new to the gravel industry but has been used for some 25 years in the coal industry. This big air operated jig does a fine job of upgrading sub-specification material. By employing a clever device to induce a partial vacuum beneath the water, and thereby give the water, *et al.*, a downward speed greater than the force of gravity alone could do, specific gravity becomes the only factor in determining the velocity of particles. This process solves the problem of larger particles, with the same specific gravity, sinking faster than light ones.

Air in concrete (L'air dans le beton)

EDMOUR CHAURET, *L'Ingénieur* (Montreal), V. 45, No. 180, Winter (Dec.) 1959, pp. 9-16
Reviewed by ARON L. MIRSKY

Broad survey of the effects of entrained air, methods of measurement, etc., with references to the recent literature and the ASTM standards. Results of some tests, both laboratory and field, conducted for the city of Montreal are also described.

Contribution to the technology of the finest fractions of the aggregates in concrete mixes (Beitrag zur Technologie des Keinkorns in Betonmischungen)

JOSEF FRITSCH, *Beton-Herstellung und Verwendung* (Düsseldorf), V. 9, No. 8, Aug. 1959, pp. 290-296
Reviewed by FERDINAND S. ROSTASY

While the importance of the finest fractions of the aggregate with respect to the qualities of concrete is well-known, it is time they are given closer attention. The basic principles of concrete technology, e.g., the relationship between the necessary amount of cement paste, water-cement ratio, and the amount of finest particles in the sand, are discussed by the author in detail. The conventional way of specifying in contracts the amount of cement, workability, and gradation is criticized, since it does not take into account the variations in the finest fractions. To obtain economic construction, safety factors have been cut down, but they have to be guaranteed. This demands a maximum attainment of the concrete qualities. This has resulted in the development, today, of aggregate preparation plants with so-called hydrosizers whose task is the removal of the dust-fine sand particles. This is not only justified from the concrete technological point of view, but also because these finest fractions are mostly of mediocre quality and have proved to be the weakest parts in the course of erosion. Several ways of eliminating these fines, such as the slurry process as used in European practice, are discussed. The rigorous control of the fine fraction yields several advantages, such as improved weather resistance, imperviousness, a reduction of cement paste, and a gain of uniformity.

Sand takes on new importance

CLYDE C. CONNOR, *Rock Products*, V. 62, No. 10, Oct. 1959, pp. 128-130, 132

Even though masonry and cement mortars have been made from identical size sand gradings, a considerable range of differences has been found to exist in their properties, such as compressive strength, volume change, durability, and workability. The reasons for these variations in properties seems to be related to the influence of voids, porosity, and pore size. The author's investigations have shown that some sand samples which produce a highly satisfactory mortar, would not have passed ASTM requirements.

Polyvinyl acetate and portland cement mortars

ROBERT T. HOWE, *Proceedings*, ASCE, V. 86, CO 1, Feb. 1960, pp. 31-40
AUTHORS' SUMMARY

Polyvinyl acetate, a synthetic resin widely used as a base for paints and adhesives, may offer possibilities as an admixture for portland cement mortars. This paper describes polyvinyl acetate, and outlines research on its possible use in a concrete floor surfacing material. Full-scale applications made several years ago are described and their present conditions evaluated.

Petrology in relation to road materials. Part 1 — Rock types used to produce aggregate

E. J. MINTY, *Journal and Proceedings*, Royal Society of New South Wales (Sydney), V. 93, 1959, pp. 27-37
HIGHWAY RESEARCH ABSTRACTS V. 30, No. 6, June 1960

During the past 6 years the Department of Main Roads, New South Wales, Australia, has received for testing over 2000 rock samples for possible use as aggregates with bitumen or cement. From the records of the physical tests

conducted on these samples details were extracted and tabulated with the object of correlating the geological features of the rocks with the properties which are of interest to the engineer.

The prime object of this paper is to illustrate the value of petrological study in assessing the potential of aggregate resources, and to bridge the gap between engineering and geological concepts related to concrete.

Guide specification for ready-mixed concrete section of concrete specifications

Concrete Committee, CSI Milwaukee Chapter, 1960 Edition, 16 pp. (1-49 copies, $1.00 each; 50-999 copies, $0.75 each; 1000-up copies, $0.50 each)

This specification covers materials used and manufactured, delivery, inspection, and testing of concrete. It makes use of industry-wide accepted standards and in general sets forth only those parts with which the architect-engineer, the architect-engineer's representative at the job site, the contractor, and the ready-mixed concrete supplier should be cognizant of in the routine execution of their work. It is intended to be used as a basis for the nucleus of a specification, and reflects the opinions and practices prevalent in the metropolitan Milwaukee area so that it should be modified to reflect local conditions where necessary. Explanation sections are included to clarify all specifications.

Control of gypsum in portland cement

Bailey Tremper, *Proceedings*, ASTM, V. 59, 1959, pp. 1101-1110 (including discussion)
Author's Summary

Progress in devising simple, practical tests to demonstrate the relationship of the SO_3 content of portland cement to its optimum value is discussed. Test methods have been patterned on the principles set forth by Lerch in 1946. These are that for each cement there is an optimum percentage of SO_3 which produces maximum strength and minimum volume change and which is related to the rate of depletion of gypsum during the early stages of setting and hardening. Emphasis is placed on results of cooperative testing for short-time expansion in water and contraction in air of mortars containing the cement in question and the same cement to which pulverized gypsum is added in the laboratory to increase the SO_3 content 0.5 percent.

Quantitative determinations of the four major phases in portland cements by x-ray analysis

Stephen Brunauer, L. E. Copeland, D. L. Kantro, C. H. Weise, and Edith G. Schultz, *Proceedings*, ASTM, V. 59, 1959, pp. 1091-1100
Authors' Summary

A method has been developed for the direct determination of the four major phases (tricalcium silicate, dicalcium silicate, tricalcium aluminate and calcium aluminoferrite) as well as of the composition of the calcium aluminoferrite phase in portland cements by x-ray quantitative analysis.

This method, which employs no assumptions about the compositions of the phase, has been compared with the previously reported combined x-ray and chemical analysis method, which does employ certain assumptions about compositions. It has been found that the results for three of the four phases are in good agreement and for one (tricalcium silicate) in fair agreement. The x-ray method results are systematically higher than the combined method results for all phases except tricalcium aluminate, for which the methods are in agreement.

A comparison of the sums of the phases with the sums of the oxides obtained from chemical data shows that the combined method gives somewhat more accurate results than the x-ray method. Using the combined method values for tricalcium and dicalcium silicate, new calibration constants have been obtained, and the x-ray method values have been recalculated for the silicates with the new constants.

Coming role of pozzolans. Part IV—Heat-activation techniques

WOLF G. BAUER, *Pit and Quarry*, V. 52, No. 12, June 1960, pp. 95-99, 134

Heat activation at relatively low temperatures can be accomplished in several types of kilns and calciners suited to the particular physical properties of the raw material.

The choice may hinge on the conservation economics as it affects complete or partial utilization of the deposit, as well as on the ability and suitability of the prepared material particles to hold up under the process without breakage or attrition and dusting.

It is also to be noted that, irrespective of the kiln or calciner type used, the basic control problem will center on the accurate maintenance of preestablished time-temperature history.

See also Current Reviews, ACI JOURNAL, V. 31 No. 10, Apr. 1960, (*Proceedings* V. 56): p. 1084; No. 11, May 1960, p. 1198.

Pavements

Rationalization of concrete base construction (in German)

WINTERNITZ, *Strassen-Asphalt- u. Tiefbau-Technik* (Heidelberg), V. 11, No. 22, 1958, pp. 728-730

ROAD ABSTRACTS Dec. 1959

Describes the use of square precast concrete slabs in constructing a base for rolled asphalt in Stuttgart.

Quality control of pavement concrete on aerodromes in new South Wales

LESLIE L. SIMON, *Constructional Review* (Sydney), V. 33, No. 6, June 1960, pp. 28-39

AUTHOR'S SUMMARY

Describes the methods used and results achieved by the New South Wales branch of the Commonwealth Department of Works in the quality control of concrete pavements on three major aerodromes.

It is stated that it was possible to reduce the variations in the quality of the concrete—as indicated by the flexural test results—to such extent that further reduction of this variability has hardly any practical value.

The paper also deals with investigations on the relationship of the 28-day and 90-day flexural strengths and with the effect of the mortar-cube strength of the cement and its variability on the flexural strength of the concrete.

Proposed method for rational design of concrete slab in highway pavement (in Japanese)

AKIRA NISHINURA, *Transactions*, Japan Society of Civil Engineers (Tokyo), V. 21, No. 65, No. 1959, pp. 37-42

Reviewed by KIYOSHI OKADA

Based on the assumption that the failure of a concrete pavement slab is caused by repeated bending stresses due to vehicle wheel loads, a formula for determining the thickness of slab is theoretically derived. This formula is a function of relative frequency of heavier vehicles, traffic volume, required service life, bearing strength of subgrade, and probability of failure of concrete.

Asphalt-surfaced and concrete pavements for airports

Proceedings, Institution of Civil Engineers (London), V. 14, Session 1958-59, Sept. 1959, pp. 19-30*

Requirements for airfield pavement surfaces

GUTHRIE S. COOPER, pp. 19-22

Asphalt surfacing for airfields

ALAN W. ATTWOOLL and DONALD C. BROOME, pp. 23-26

Suitability of concrete for airfield runways

ARTHUR W. HILL, pp. 27-30

Reviewed by ARON L. MIRSKY

First paper presents a general comparison of asphalt and concrete airfield pavements, under 12 headings ranging from economy of construction to resistance to aircraft jet temperatures to suitability for strengthening. Second and third papers discuss many of these topics in more detail. Orientation is primarily to British needs and practices.

*Discussion of all three papers is printed in the issue of May 1960, pp. 71-89.

Deformation of the railway track on concrete pavement under vertical load (in Japanese)

YUTAKA SATOH, *Transactions*, Japan Society of Civil Engineers (Tokyo), V. 21, No. 65, Nov. 1959, pp. 56-62

Reviewed by KIYOSHI OKADA

Describes the results of loading tests made on railway track laid on concrete pavement. Settlement of the track and stresses in concrete pavement, due to the test loads, were in good agreement with the theoretical values obtained by the author.

Epoxy resin for structural repair of concrete pavement

WILSON L. DAVIS and EUGENE PINKSTAFF, *Proceedings*, ASCE, V. 86, CO 1, Feb. 1960, pp. 55-70

AUTHORS' SUMMARY

Describes tests performed in the North Central Division Laboratory of the Corps of Engineers in formulating epoxy resins that will satisfactorily repair surface defects in portland cement concrete pavements, that will bond the hardened surfaces of concrete together, and that will bond new concrete to existing concrete. It describes also the field application of the resulting epoxy resins.

Slash curb costs

Construction Equipment, V. 20, No. 4, Oct. 1959, pp. 93-95

The cost of curbs on highways has been reduced in Texas from $1.50 per linear ft to 45-50 cents. This was accomplished by employing a self-propelled slip-form curb paver, thereby eliminating forms, labor crews, filling forms, and movement of equipment.

Novel spreader sparks central mix paving job

Roads and Streets, V. 102, No. 12, Dec. 1959 pp. 58-60, 67

Describes the successful operation of a fully automatic job site batch plant which supplied concrete via ready-mixed concrete trucks to a self-propelled highway pavement spreader. This batch plant was situated between two contiguous contract sections and provided an average of 165 cu yd per hr to the paving sites. The pavement spreader has an adjustable screed and deposits the mix on the base as fast as the truck can discharge, this is facilitated by means of a swinging conveyor belt.

Where states stand on central and transit mix

Roads and Streets, V. 103, No. 3, Mar. 1960, pp. 113-115

A state-by-state survey of highway departments on the acceptance or non-acceptance of either transit-mixed or central-mixed concrete. A large majority of states allow either type of mix, while only seven states reject either method. Reasons for prohibiting the above methods by some states are given.

Tentative recommendations for the construction of runways in airports (Richtlinien fur den Bau von Betondecken)

K. EBERLE, *Beton-Herstellung und Verwendung* (Düsseldorf), V. 10, No. 2, Feb. 1960 pp. 61-67

Reviewed by FERDINAND S. ROSTASY

These recommendations have been worked out by a subcommittee of the German Society of Highway Research and deal in their introduction with the various factors of runway design: loads, density of traffic, distribution of traffic, the different load cases occurring during the operation of airplanes. Following the introduction items as the subconstruction, design of runway comprising the concrete to be used, reinforcement of the slabs, joints, construction of the slab, supervision and necessary field tests regarding evenness and concrete strength are discussed.

Design and construction of concrete pavement (in Japanese)

AKIO OKUDA, *Transactions*, Japan Society of Civil Engineers (Tokyo), V. 20, No. 59, Extra Papers 3-2, Dec. 1958

Reviewed by KIYOSHI OKADA

Crack occurrence and causes of deterioration of existing concrete pavements are statistically analyzed with relation to various factors such as width, thickness, length of slab, type

of joints, traffic conditions, and subgrade conditions. Results of strength tests on drilled cores are also studied statistically. Based on an extensive survey of construction practices, the author points out some interesting findings with regard to the location of mixing plants, proportion of concrete mixture, and the method of quality control of placed concrete.

Precast Concrete

Block kiln arrangement — Good design proves profitable

WILLIAM J. SHORE, *Modern Concrete*, V. 24, No. 2, June 1960, pp. 52-55

An analysis of the design and workable performance of some block kiln layouts observed shows evidence of their having been copied from those which might have been considered proper in the 1930's. Author presents a layout which offers a saving of one-third in initial construction cost plus savings in operation.

Casting 2000 sculptured panels for Chicago Exposition Center

Concrete, V. 67, No. 10, Oct. 1959, pp. 18-20

Chicago's Exposiiton Center uses sculptured concrete wall panels to cover a 185,000 sq ft of building surface. A total of 2010 wall panel sections, averaging 17 x 4½ ft and weighing 2½ ton, were precast for the job. Of this total, 152,000 sq ft are sculptured and geometric design panels in bas-relief. The panels use exposed aggregate of white quartz set in white cement.

Concrete railway sleepers

D. H. LEE, *N. Z. Concrete Construction* (Wellington), V. 4, No. 4, May 1960, pp. 52-57
AUTHOR'S SUMMARY

Discusses in a general way the development of flat-bottomed rail track and the use of timber, steel, and concrete ties. Also reports on long welded rails and important advantages to be gained when used with concrete ties. Recent experience in three countries in connection with resilient pads under the rail and elastic fastenings is also mentioned. Practice and opinions in regard to prestressed concrete and reinforced pad type ties are discussed.

Precast facing panels have exposed aggregate

Contractors and Engineers, V. 56, No. 12, Dec. 1959, pp. 66-68

Describes the facing of the exterior and inside columns of a hotel. The entire exterior is covered with either precast concrete window panels having exposed aggregate or, where the exterior walls are flat, with flat skin panels bolted directly to the concrete frame. Interior columns are faced with polished precast panels which resemble marble.

Permeability and absorption of precast concrete products

M. LEVY, *Civil Engineering and Public Works Review* (London), V. 55: No. 642, Jan. 1960, pp. 88-94; No. 643, Feb. 1960, pp. 248-251
AUTHOR'S SUMMARY

A theory of permeability and absorption for precast products, or for that matter any material of similar pore structure, has been enunciated. This theory, based upon the empirical Poiseuille equation, is supported by the experimental evidence from permeability tests for well over 100 different precast units. It is concluded that since the theory enables predictions to be made from short term behavior then all the information required can be obtained from two readings, one at 10 min and one at 1 hr; no test as long as 24 hr duration is necessary.

The established empirical permeability equation is:

$$p = at^{-n}$$

where p is the permeability in $ml \times 10^{-6}$ per sq cm per sec at any time up to percolation; t is the time; a is a constant; n is a parameter varying from 0.3 to 0.7 (0.5 theoretically).

The above mentioned two readings not only enable one to determine a and n, but by the use of nomograms one can estimate when water will percolate out of the back of a unit. The

value of n is constant for a particular mix design, method of manufacture, and so on. Deviations of n either side of the theoretical value of 0.5 are thought to be due to the processes of silting up and flushing through of pore capillaries and further experiments are suggested to test this theory.

The empirical absorption equation is the integral of the above equation and although no experimental data is available it is felt that two short term absorption results should prove or disprove this formula. Longer term absorption readings up to 24 hours are of little use as they are asymptotic in value and little or no more water can be absorbed. This is thought to be due to a pocket of air being trapped at the center of the specimen and limiting the ultimate figure. It is evidently the rate at which water is taken in and not this final figure which is important and this explains why the 10 min absorption is at present the best criterion as a guide to durability.

Prestressed Concrete

Construction of prestressed concrete water tank (in Japanese)

ISAO IBUKA and KAZUO WATANABE, *Journal*, Japan Society of Civil Engineers (Tokyo), V. 44, No. 9, Sept. 1959, pp. 15-20
Reviewed by KIYOSHI OKADA

Describes the construction, in detail, of prestressed concrete water tanks in Yokohama City. The tank, 17.20 m high and with an inside diameter of 8.0 m, is to be used to adjust the static pressure exerted on the distributing pipe of city supplied water. Freyssinet cables were used for both longitudinal and circumferential prestressing.

Compliance test of prestressed concrete (Une application eprouvee du beton precontraint)

CLAUDE LANTHIER, *L'Ingénieur* (Montreal), V. 45, No. 180, Winter (Dec.) 1959, pp. 31-34
Reviewed by ARON L. MIRSKY

Illustrated report of a prestressed double-tee slab, tested in the structural laboratory of the Ecole Polytechnique, Montreal, for the Pressure Pipe Company of Canada, Ltd. The slab, designed in accordance with Chapter 11 of the "Bylaw Concerning the Erection of Buildings in the City of Montreal (No. 1900)," withstood without failure the loading recommended in the ACI-ASCE Committee 323 Tentative Recommendations for Prestressed Concrete (ACI JOURNAL, V. 29, No. 7, Jan. 1958 (*Proceedings* V. 54), pp. 545-578.

Much the same data (in English), with additional photographs, will be found in the *Journal* of the Prestressed Concrete Institute, V. 4, No. 2, Sept. 1959, pp. 17-22, under the title "Test of a Prestressed Concrete Double-Tee Slab."

Prestressed concrete wharf construction at Hobart

H. NEWTON, *Constructional Review* (Sydney), V. 33 No. 5, May 1960, pp. 24-28

Describes precasting and erection of precast-prestressed piles and wharf decking for two wharf berths.

Creep and prestressed concrete (in French)

J. DELARUE, *RILEM Bulletin* (Paris), No. 4, New Series, Oct. 1959, pp. 32-52
AUTHOR'S SUMMARY

Some tests were performed to compare shrinkage and creep of the same prestressed concrete, on a bridge beam 28 m long and on prismatic samples, 1.0 m long, maintained in air with relatively constant humidity. In the laboratory tests, influence of transverse dimensions and of age of concrete at loading time were studied. Shrinkage strains and creep strains were always dissociated.

Shrinkage progresses quite differently in the two cases; for the laboratory samples, it varies in a classical way; on the beam, in place, after a steep start, strains vary between two extreme values, with swelling in winter, shrinkage in summer.

In the progression of creep, two steps can be distinguished: (1) during the first step, strains are in direct

proportion to the square root of time, for the beam as well as for the laboratory samples. The younger the piece at loading time or the greater transverse dimensions, the longer is this step. Then, every law of shrinkage seems to be available for creep. (2) Later, in the second step, creep runs in a parallel way for all laboratory samples which would keep from their youth only a constant difference. The beam creep runs a little slower but, finally, creep strains of equally stressed fibers, on beam and samples, appear to be of the same order of magnitude.

Prestressed concrete in marine structures

BEN C. GERWICK, *Civil Engineering*, V. 29, No. 11, Nov. 1959, pp. 46-49

Reviews the problems encountered in such marine structures as wharves, piers, dolphins, and mooring platforms and their constituent structural elements such as piles, caps, deck slabs, beams, and girders.

Portable prestress bed saves $30,000 trucking costs

Construction Equipment, V. 21, No. 6, June 1960, pp. 31-35

Describes the design and operation of a portable prestressing bed put into operation by a contractor in an attempt to reduce costs of buying ready-made prestressed beams and piles. The bed consists of box beams, 30 ft long and 3½ ft square, layed side by side and end to end for a length of 286 ft. The boxes are held straight and true by stress rods that are hooked together and placed in four pipes set in the sidewalls of each box; they are then stretched to 90,000 lb tension which squeezes the boxes together and makes them into one individual unit. The boxes are then covered with 3 in. of concrete which is the last step in forming the prestressed bed. The operation of building prestressed precast beams and piles then begins. Two techniques worth mentioning are the use of fiber tubes (saves concrete by forming a hollow center core in the pile) and single-point jacking (on this job it proved more efficient then multiple jacking).

Prestressed bridge beams—V. 1

KENNETH K. KIENOW and JOHN B. SCOTT, Western Technical Publications, Tucson, Ariz., 40 pages, $6

Design data are presented for 67 precast, prestressed highway bridge sections, including solid slab, cored slab, and box girders, noncomposite, and with 3- and 4-in. composite slabs. All pertinent design information including minimum prestressing force, number of ⅜-in. strands, eccentricity, mild steel area, gross weight of section, and #4 stirrup spacing has been tabulated for 2-ft increments of span for each section. Both 36 and 48 in. wide sections are included, and depth of sections varies from 12 to 48 in. The sections studied cover the range of spans from 16 to 114 ft. Designs tabulated include both 5000 and 6000-psi beam concrete. Live and dead load moments and shears, section properties, ultimate moment, and load fiber stresses have been tabulated for each design. The tables are meant to serve as a guide in the selection of the optimum section and not as a substitute for a detailed analysis.

Design and construction of a prestressed concrete box beam highway bridge (in Japanese)

YASUJI TAHARA, *et al.*, *Transactions*, Japan Society of Civil Engineers (Tokyo), V. 21, No. 60, Extra Paper 3-1, Feb. 1959
Reviewed by KIYOSHI OKADA

Describes in details the design and construction of a prestressed concrete box beam bridge built in 1956 near Osaka. The length of span is 30.40 m and the depth-span ratio is 1:27.6. Results of several tests, conducted with a small plastic model, and on a prestressed model of ⅕ scale, are also presented. Loading tests and deformation measurement due to creep and shrinkage of concrete were made after the completing of the bridge.

Congelation in cable ducts of prestressed concrete structures (in Danish)

ERIK KALHAUGE, *Nordisk Betong* (Stockholm), V. 4, No. 2, 1960, pp. 109-117
Reviewed by MARGARET CORBIN

Congealing of water (or mortar) in cable ducts of a prestressed concrete structural unit produces cracking and other damage to the concrete due to the high pressures which occur even at a few degrees C below 0 C. Real danger sets in when the temperature drops suddenly to −10 C or lower. Freezing of water prior to injection can be prevented by blowing the cables out with compressed air. The use of steam for this purpose carries the risk of condensation and should be avoided. Instructions are given for electrical heating of frozen cables. Congealing of mortar in the cable ducts can be prevented by keeping the temperature of the structure above zero (C) for a week after injection—with a water-cement ratio equal to 0.35 to 0.40.

Stresses permissible in prestressed concrete

P. W. ABELES, *Concrete and Constructional Engineering* (London), V. 55, No. 2, Feb. 1960, pp. 73-81

The stresses recommended in the new British Standard Code No. 115 (1959), "The Structural Use of Prestressed Concrete in Buildings," are discussed and compared with the corresponding stresses given in the German provisional recommendations (DIN 4227-1953) and those stresses given in the tentative recommendations of ACI-ASCE Committee 323 [ACI JOURNAL, V. 29, No. 7, Jan. 1958 (*Proceedings* V. 54), p. 545].

The stresses covered are: compressive stresses permissible at transfer, compressive stresses permissible at working load, tensile stresses permissible at working load, temporary tensile stresses at transfer and during transport, principal tensile stresses, permissible stress in bending in concrete at load causing failure, stress permissible in steel at transfer and working load, stress due to bending permissible in steel at load causing failure, permissible stresses in reinforcement resisting shearing, and stresses permissible in composite members.

Precast concrete for winter building

C. R. CROCKER and A. W. SMITH, *The Engineering Journal* (Montreal), V. 42, No. 11, Nov. 1959, pp. 61-65
Reviewed by ARON L. MIRSKY

To reduce cost and delays of winter concrete construction in Canada, use of precast concrete (simple, prestressed, liftslab, etc.) was investigated. Paper discusses history, practices in other countries, factory versus on-site manufacture, simplification of construction operations, the problem of connections between precast units, and —briefly — current Canadian practice.

Problem of nonlinear creep of concrete in prestressed structures (in French)

Z. BYCHAWSKI, *RILEM Bulletin* (Paris), No. 4, New Series, Oct. 1959, pp. 53-58
AUTHOR'S SUMMARY

The aim of this paper is to show the applicability of the nonlinear creep theory to prestressed concrete and to give a comparison of nonlinear creep effects with the linear theory conclusions.

The theory of distortions generalized by introduction of time parameter was applied. In the case of a statically determinate beam, the hypothesis of plane sections was admitted. Considering the general case of stresses variable with time and admitting the nonlinear function of stress as a polynomial, the author gets to the system of two nonlinear integral equations for two functions of time tc be found. As an example an axially prestressed and an axially charged element were considered.

The final conclusion with some experimental data is that the nonlinearity of creep is characterized by the loss of 8 and 20 percent more of tension

in reinforcement than in linear creep theory. The coefficient of nonlinearity increases as the prestressing increases; it provokes an increase of tension loss in the reinforcement.

This leads to the conclusion that when evaluating the loss of tension in the prestressed reinforcement, the conditions of work of concrete have to be taken into consideration; in the case of high pressures, the nonlinearity of creep.

These theoretical results are presented in diagrams.

Profiled wires as used in prestressed concrete (in Polish)

T. Basiewicz, *Inzynieria i Budownictwo* (Warsaw), No. 8, 1959, pp. 333-340
Polish Technical Abstracts No. 1 (37) 1960

When designing a new type of concrete railway ties, the possibility was investigated of increasing the adhesion to concrete of wires of 2.5 mm diameter, twisted in pairs. Results are presented of experiments carried out with profiled wires in Switzerland, the USSR, Hungary, and France; experiments made in Poland are discussed more fully, taking into consideration the shape of the profile, the adhesion of the profiled wires to concrete, and their technology. Test results concerning mechanical and rheological properties of profiled wires are also given.

Prestressed concrete railroad ties

Roy L. Peck, *Modern Concrete*, V. 23, No. 6, Oct. 1959, pp. 48-52

Concrete railroad ties have been tested by railroad companies for over 60 years, but it is only recently that they have been considered as a real possible substitute for wood ties, this is due primarily to the potential possibilities of prestressing. At present the American Railroads Research Laboratories, Chicago, are conducting tests on five differently designed prestressed ties. These tests were conducted to investigate several important factors such as: maximum positive and maximum negative bending moments, best type of aggregate, over-all weight, and minimum concrete strength. It is estimated that concrete ties may be competitive with timber ties if they can be made for $9 or less.

Properties of Concrete

Relation between total and local compressive strength of cylindrical mortar specimens (in Japanese)

Takezo Kodama and Mitsuo Ugai, *Semento Gitjutso Nenpo*, V. 12, 1958, pp. 374-377
Ceramic Abstracts Feb. 1960

Cylindrical mortar specimens 5 x 10 cm with water:cement: sand = 0.65:1:2 were cured in water for 28 days, and the compressive strength was determined by placing the specimens between compressive cylinders 1 to 5 cm lower bearing blocks. A linear relation was found between the ratio of local to total compressive strength and the reciprocal of the diameter ratio of the compressive cylinders.

Experiments on the influence of the mineralogical character of aggregates on the creep of concrete

K. Kordina, *RILEM Bulletin* (Paris), No. 6, New Series, Mar. 1960, pp. 7-22
Author's Summary

Eight different concretes were examined which differed only in the kind of aggregates. The ratio of the mixture — with two exceptions — and the storage conditions were the same in all experiments. Measurements of elasticity, shrinkage, and creep were carried out with cylindrical concrete specimens 20 x 80 cm in the longitudinal and transverse direction. In addition, attempts were also made to determine, as far as possible, the respective properties of the aggregates so that the reasons for the observed differences in the behavior of creep of the eight concrete mixtures could be found. The creep specimens were submitted to load 28 days after batching. At this age the stress was equal about to ⅓ of the cube strength and the experiments were carried out in a cli-

matically conditioned room at a temperature of about 20 C (68 F) and a relative humidity of about 65 percent. Experiments lasted between 1 and 2 years. The deformations due to creep showed pronounced differences; with ordinary aggregates the ratio between them being approximately 1:1.5.

The elastic deformations of the concretes examined were also different, but their ratio did not correspond to the deformations caused by creep.

Concrete in chemically aggressive water (Beton in chemisch angreifenden Wassern)

K. SEIDEL, *Beton-Herstellung und Verwendung* (Düsseldorf), V. 9, No. 10, Oct. 1959, pp. 364-365

Reviewed by FERDINAND S. ROSTASY

Reports on behalf of the German Reinforced Concrete Committee tests and inspections regarding the chemical resistance of concrete. Regardless of the cement type it was found if the cement dosage was 7.2-8.9 sacks per cu yd and the water-cement ratio 0.35-0.50 that the concrete remained unattacked by seawater for 50 years, even without the use of pozzolans. The same observations were made for concrete stored in moor water. Concluding it is said that density and uniformity of the concrete are of greater importance for the resistance than the cement type. Only in lean mixes, and/or if highly aggressive water is present, the cement type is of importance. For this case sulfate resistant cements have been developed in the last years.

Influence of age on aluminous cement concrete (in French)

V. MATIC, *RILEM Bulletin* (Paris), No. 6, New Series, Mar. 1960, pp. 33-38

AUTHOR'S SUMMARY

To ascertain the initial point and the end point of the decrease in the strength of aluminous cement concretes and to calculate the order of magnitude of this decrease, compressive tests are being carried out on a large number of test pieces.

The rupture ages extend from 3 to 20 years. Previous tests show that the maximum strength is reached at 3 months, the decrease appearing only subsequently. Final results will be known in 1969.

At present, results seem to show that the strength of this concrete decreases after 90 days and that this decrease continues practically over the first 5 years.

The maximum decrease is approximately 50 percent in relation to the strength at 90 days. It seems that this decrease ceases after 5 years and that from this point on an increase in strength is to be noted. This last observation, however, will be confirmed only after the results of rupture tests to be made at 10 years of age have been obtained.

Effect of creep in hyperstatic concrete structures subjected to different conditions of stress (in French)

F. LEVI, *RILEM Bulletin* (Paris), No. 5, New Series, Dec. 1959, pp. 18-25

AUTHOR'S SUMMARY

Describes creep tests carried out on two-hinged arches provided with checking devices by which to measure the displacements of the supports and the crown, as well as the deformations of the extreme fibers and of the mean fiber along the whole length of the arch. The values of the shrinkage, of the creep and of the thrust in relation to time are determined by this means.

Test pieces are subjected to discontinuous distortions. It then seems that creep is greatly increased by the increase in the stresses created by the return of the thrust to its initial value, and from this it is deduced that the laws of linear creep would remain valid in the presence of variable external loads.

From the practical point of view, these tests may be considered as representative of what happens in numerous structures that undergo a succession of modifications in the bonding conditions during their placing (arches with provisional hinges, floors bearing previously in one direction and suc-

cessively transformed into slabs, etc.). Independently of the effect of shrinkage, the bondings introduced after placing may be an appreciable factor.

Perlite concrete—density and compressive strength in relation to mix proportions and methods of curing

F. A. Blakey and E. N. Mattison, *Report* No. C3 1-1, Division of Building Research, Commonwealth Scientific and Industrial Research Organization, Melbourne, 1958, 26 pp.
BRI Abstracts June 1959

Effect of four curing treatments on the compressive strength and density of perlite concretes of differing mix proportions was studied. Results show that the strength and density of the concrete decreases with increasing perlite content and with the use of air-entrainment. Unexpectedly low strengths were obtained with concrete cured in high pressure steam, and possible explanations for this behavior are noted.

Investigations into the strength of concrete under sustained load

R. Sell, *RILEM Bulletin* (Paris), No. 5, New Series, Dec. 1959, pp. 5-13
Author's Summary

The influence of a continual, steady load on the strength and deformation of concrete was shown on a series of 112 specimens of "B-300" concrete. The age of the concrete when loaded and the eccentricity of load were varied.

The tests show that ultimate strength under sustained load determined on prismatical specimens is about 30 percent lower than the short-time ultimate cube strength. The deformations at ultimate stress under sustained load are 2-4 times as extensive as those at short-time ultimate strength.

Influence of sustained loads on the strength of concrete

J. Glucklich, *RILEM Bulletin* (Paris), No. 5, New Series, Dec. 1959, pp. 14-17

Since concrete has the property of delayed elasticity, its energy content is increasing exponentially with time when it is subjected to a sustained load. Its strength, as expressed by a stress-limit, is therefore a decreasing function of the loading time. This feature has not received adequate attention due to the fact that when concrete members form parts of structures these are nearly always statically indeterminate systems, while when concrete is tested in a testing machine the latter is nearly always of the "hard" type. As in such cases, the starting of internal cracking is affected but not the final fracture, the effect is thus usually not observed. The introduction of a spring between the specimen and the testing machine reveals its presence.

Influence of age on strength—Some tests carried out with cement having a high initial strength (in French)

D. Jevtic, *RILEM Bulletin* (Paris), No. 5, New Series, Dec. 1959, pp. 41-48
Author's Summary

First part of this report deals with the study of the increase in strength of concrete in terms of the water-cement relationship. It would seem that the water-cement relationship has no influence on the strength of concretes whose age exceeds 28 days. Measurement of deformation taken before crushing led the author to propose an expression for E for secant modulus corresponding to the stress based on a relationship between modulus of the tangent initially (dynamic modulus) and prism resistance to rupture by compression.

Influence of temperature on the creep of mass concrete

J. Laginha Serafim and M. Q. Guerreiro, *RILEM Bulletin* (Paris), No. 6, New Series, Mar. 1960, pp. 23-32
Authors' Summary

The equipment, set-up, and instruments used are summarily described. The tests were carried out on mass-cured concrete prisms, 20 x 20 x 60 cm, protected by copper jackets. Twenty-two prisms were cast, separated in two stages. Four prisms of the first stage kept at room temperature and four others at 45 C were loaded at 3 days

with a stress of 15 kg per sq cm; unloaded control prisms were kept also in the same conditions. In the second stage of the test 8 day old prisms were loaded at 25 kg per sq cm, some kept at 45 C, others kept at room temperature in mass-cured and saturated conditions. A mass-cured prism was also loaded at 300 days.

The principal conclusion drawn from these tests is that the influence of a high constant temperature on the creep strains of concrete 3 to 8 days old when loaded seems to be slight, being felt only in the period of 4 to 7 days following loading. Thereafter the rate of creep is approximately the same for both temperatures.

Influence of the finest fraction of the aggregate upon the qualities of concrete (Der Einfluss des Feinstkorns auf die Eigenschaften des Betons)

W. SCHULZE, *Beton-Herstellung und Verwendung* (Düsseldorf), V. 10, No. 2, Feb. 1960, pp. 45-52

Reviewed by FERDINAND S. ROSTASY

The amount of the finest fraction of the aggregate which lies according to the German specification between 0 and 0.02 mm (0.0008 in.) is of influence with respect to the necessary amount of water, necessary amount of cement, workability, and frost resistance. The author reports tests and discusses their findings. It was found that finely ground stone powder of 0 to 0.1 mm grain size improves workability, strength, and imperviousness. This is especially valid for lean mixes. Frost resistance decreased. However, air entrainment may make up for this disadvantage.

Tension bending test: Study on the influence of the speed of loading (in French)

DUHOUX and TESSIER, *RILEM Bulletin* (Paris), No. 6, New Series, Mar. 1960, pp. 47-52

AUTHORS' SUMMARY

The tests involved putting concrete prisms 14 x 14 x 56 cm through the bending test under constant moment, taking the speed of loading as the sole parameter. Five rates of loadings were adopted, scaled between 0.15 and 4 kg per sq cm per sec.

Before the rupture test, each test piece was given a sonic test to determine its dynamic modulus of elasticity, each series of tests used ten test pieces. The moduli of elasticity of the various test pieces are within a narrow range, for the coefficient of variations overall is but 1.4 percent.

The bending results are more scattered since the coefficients of variations of the different series vary from 4.9 to 8.3 percent, but one can but observe a greater scatter for the higher speeds.

The rupture stress increases with speed. By making the speed vary in the 1 to 20 relation, the rupture rate has been increased by about 15 percent.

Finally, the relation between the dynamic modulus and the bending strength of a concrete is not absolutely constant, however, the relation $E_D = 70{,}000 \sqrt{R_t}$ proves to be correct within 10 percent.

Effect of internal vibration of concrete mixtures upon the entrained air

F. H. BLANDIN and O. LARSEN, *Illinois Highway Engineer*, V. 11, No. 2, 1959, pp. 2-4, 10

HIGHWAY RESEARCH ABSTRACTS Nov. 1959

In accordance with the requirements of the "Standard Specifications for Road and Bridge Construction," concrete produced for the Illinois Division of Highways must have an entrained air content of 3 to 5 percent of its volume. For concrete in its plastic state, it is relatively simple to see that this requirement is complied with. However, the work performed on the concrete during placement is known to remove some of the air, and some apprehension exists as to whether the hardened concrete may at times be left with substantially less than the desired amount of entrained air.

Since the development of the high pressure method for determining air

contents of hardened concrete, some consideration has been given to the feasibility of conducting an investigation to study the loss of entrained air by internal vibration. It was decided, however, first to make a few pilot tests to simulate the loss in air content as related to period of vibration, and to determine if the air that remains in the hardened concrete is sufficient to prevent reduction in durability. With this object in view, a number of small batches of concrete were prepared and, after various periods of internal vibration, were made into specimens for testing in alternate freezing and thawing with intermittent determinations of their dynamic moduli of elasticity to observe their degree of deterioration during the tests.

Although it would be difficult to correlate the degree of vibration imparted to the small batches of concrete with that obtained in actual practice at various locations in structures, the conclusions as stated below probably are generally applicable to mixtures initially having air contents of 4 percent.

(1) In mixtures of dry consistency, say less than 2-in. slump, it appears unlikely that the entrained air content will be reduced below 3 percent by a reasonable amount of internal vibration.

(2) In mixtures of wet consistency, say more than 5-in. slump, it appears that the entrained air content may easily be reduced below 3 percent by even a moderate amount of vibration.

(3) The results of the durability tests indicate strongly that when the air contents are reduced below 3 percent, the loss includes some of the air that is beneficial in imparting durability to the concrete.

(4) The resistance of concrete to freezing and thawing, regardless of air content, is reduced by an increase in the water-cement ratio, as would occur through arbitrary addition of mixing water to increase slump.

(5) It may be inferred from certain ASTM methods of conducting freezing and thawing tests that durability factors of 70 at 200 cycles, and 60 at 300 cycles of the test, are reasonable low limits for the durability factor. If this be taken at a criterion, the data indicate strongly that inferior resistance will result if the entrained air by reason of vibration is reduced below 3 percent.

(6) The results of the tests undoubtedly are of sufficient interest to warrant further investigation.

Structural Research

Some problems in concrete construction

A. D. Ross, *Magazine of Concrete Research* (London), V. 12, No. 34, Mar. 1960, pp. 27-34

Describes briefly five minor investigations. Only the salient points of each investigation are mentioned: (1) stresses in the end-block of a post-tensioned beam; (2) creep of concrete containing fly ash; (3) creep and relaxation of prestressing wires; (4) spiral reinforcement in short concrete columns; and (5) friction in prestressing.

Effectiveness of stirrups as shear reinforcement in reinforced concrete beams

M. Sekhon and A. S. Hall, *Constructional Review* (Sydney), V. 33, No. 4, Apr. 1960, pp. 33-36

Presents the results of tests performed to determine the effectiveness of stirrups and their contribution to shear strength. In a number of beams, one stirrup was placed at different locations of the shear spans so that the contribution to the shear strength of the beam could be determined. The test results are tabulated and it appears that the effectiveness of a single stirrup is related directly to its position in the stress field. If the stirrup is located at the point of maximum principal tension, it performs to best advantage. As it moves away from this position its efficiency declines.

It is usually assumed that the stirrup effectiveness is supposed to be uniform

over the whole shear span, whereas these tests show that near the support it is not effective at all.

Review of research on composite steel-concrete beams

IVAN M. VIEST, *Proceedings*, ASCE, V. 86, ST6, June 1960, pp. 1-21
AUTHOR'S SUMMARY

Investigations of composite beams carried out during 1920-1958 in the United States and abroad included both experimental and theoretical studies. Tests of specimens with and without mechanical connectors are summarized, specimens briefly described, and major results cited. Presentation of theoretical studies emphasizes the basic assumptions of both elastic and ultimate strength theories.

Dynamical compressive deformation and failure of concrete under earthquake load (in Japanese)

TADASHI HATANO and HAJIME TSUTSUMI, *Transactions*, Japan Society of Civil Engineers (Tokyo), V. 22, No. 67, Mar. 1960, pp. 19-28
Reviewed by KIYOSHI OKADA

Describes the results of dynamic compression tests of concrete and mortar. Stress-strain relationships and the ultimate strain of concrete and mortar, subjected to a dynamic load simulating earthquake action (the failure occurs 0.03-100 sec after loading), were measured. In analyzing the mechanical behavior under loading, the Kelvin body model was introduced and the coefficients of the body were estimated.

Stresses in deep beams subjected to central and third-point loading

M. E. RAVILLE and F. J. MCCORMICK, *Proceedings*, ASTM, V. 59, 1959, pp. 1230-1236
AUTHORS' SUMMARY

Many workers have noted that values of the modulus of rupture for concrete when computed from tests on beams subjected to a central load are consistenly different from values based on beams subjected to third-point loading. Attempts have been made to explain this difference. In this paper it is shown that for beams so loaded but subjected to the same maximum bending moment, the maximum longitudinal stresses at the lower surface differ significantly. All results are based on the elastic theory, as is the modulus of rupture formula.

General

Laboratory of concrete technology (in Swedish)

ARVO NYKANEN, *Nordisk Betong* (Stockholm), V. 4, No. 2, 1960, pp. 87-100
Reviewed by MARGARET CORBIN

Article describes facilities and touches on programs of research of the State Institute for Technical Research (Finland). The concrete laboratory, at Otanieni, about 9 km from Helsinki, was completed in 1958. It is a three-story building with 8000 cu m of space. Subjects under study are: winter concreting; concrete for shielding; corrosion of concrete and steel; admixtures concrete mixers. The results are published mainly in Finnish, Swedish, or English, and appear as a series of publications issued by the State Institute for Technical Research.

Dictionary of civil engineering and construction machinery and equipment (English-French)

HERBERT BUCKSCH, Editions Eyrolles, Paris, 1960, 420 pp., 39.40 F.

This is a "one-way" dictionary to translate from English to French. It has been compiled in an attempt to foster the harmony of international partnership in civil engineering and the latest relevant literature has been searched for its compilation. Subjects covered by terms in the dictionary are road and airfield construction, soil mechanics, dams, harbors, tunnels, foundations, bridges, prestressed concrete, construction materials, etc. The coverage appears to be extremely complete and includes not only words but also terms and phrases. This dictionary will be especially helpful for individuals in translating letters and reports from English to French and selecting the terminology to transmit the proper meaning.

ACI NEWS LETTER

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memo from the President:

So You Want More Papers on Construction

In a recent readership study of the ACI JOURNAL, Managing Editor Bob Wilde found that the five most-read types of papers are, in order

Construction practice
Structural design — Construction
Structural design
Structural research
Materials research

This was also approximately the order of preference for papers named by members as being valuable in their work. The high preference for papers on construction is not so surprising if we consider that, next to design, more of our members are engaged in construction than in any other single activity. Numerous suggestions brought out by the survey include "We need more articles on construction methods and practices;" "Closer contact with field practice — new methods and products being tried;" "New ideas in good economical construction details;" "More detailed data on design and construction practices in bridge construction field;" "Detailed reports on successful construction practices in new areas such as thin shell, folded plate, tilt-up, lift slab, etc;" "More articles devoted to making and using concrete;" "More articles on actual construction problems, methods, etc., by construction superintendents, batch-plant operators, and others engaged in the field;" "Keep trying for more construction articles."

The need is definite and evident, but it is not new. For years the editors and TAC have scouted for construction papers, and they would gladly give them preference in publication.

The problem lies in the innate shyness or busyness or laziness of construction people which makes them reluctant to write about their work. "There it is," they say, "it speaks for itself." It does, to the limited number who are close enough to see the structure directly. But to other thousands an illustrated account of the techniques, tricks of the trade, and troubles would be fascinating and useful.

So, if you on the firing line want to see more JOURNAL papers on construction, build a little fire under your friends (including yourself) who have the makings of a good story but don't realize it or who think they haven't the time or the ability to write it. Straightforward writing is all that's needed — a general description of the structure, the special features, the problems of design or concreting, and the progress of the work. The ACI editorial staff will take care of the styling and reproduction. No special application form is necessary; either send in the manuscript (three copies) or, if you would like a preliminary opinion, write headquarters an abstract. Information on preparation of manuscripts is given in the biennial ACI Directory.

Your pay? The satisfaction of a job well done, and the silent thanks of thousands of readers. Isn't that enough?

Joe W. Kelly
President

ACI to publish monographs

The Institute plans to publish a series of monographs supplying information on specialized subjects in the concrete field and methods of practical application of the information or data.

The Technical Activities Committee has appointed a Committee on Monographs to expedite the development of this venture. TAC member G. E. Burnett, assistant chief research engineer, U. S. Bureau of Reclamation, Denver, is chairman of the committee. He will be assisted by an eight-man committee comprised of ACI Director A. Allan Bates, vice-president, Research and Development, Portland Cement Association, Skokie, Ill.; Past President Raymond E. Davis, professor, director emeritus, Engineering Materials Laboratory, University of California, Berkeley, Calif.; ACI Standards Committee member Thomas B. Kennedy, chief, Concrete Division, Waterways Experiment Station, Corps of Engineers, U. S. Army, Jackson, Miss.; TAC chairman Clyde E. Kesler, professor, Department of T. & A. M., University of Illinois, Urbana; Past President Douglas E. Parsons, chief, Building Technology Division, National Bureau of Standards, Washington, D.C.; ACI Vice-President Raymond C. Reese, consulting engineer, Toledo, Ohio; Chester P. Siess, professor, Civil Engineering Department, University of Illinois, Urbana; and ACI Vice-President Lewis H. Tuthill, concrete engineer, Division of Design and Construction, California State Department of Water Resources, Sacramento.

In their endeavor to expedite publication of the monographs, the committee will consider the available literature and determine the need for brief authoritative treatises on specialized segments of knowledge in the concrete field. The committee will select and negotiate for the writing of the texts and moderate the critical review of and approval of the finished work for publication.

The regular review channels for JOURNAL papers will be used to evaluate monograph manuscripts. Technical committees may be asked to review monograph manuscripts on subjects

continued on p. 20

'61 Slate Announced

President--Lewis H. Tuthill

Vice-President--Roger H. Corbetta

The ACI Nominating Committee, headed by M. S. Ketchum, Jr., Ketchum, Konkel and Hastings, Denver, has announced candidates to be submitted to member ballot. The elected officers will be installed at the 57th annual convention, February 20-23, 1961, at the Chase-Park Plaza Hotel, St. Louis, Mo.

For President

Lewis H. Tuthill, concrete engineer, Division of Design and Construction, California State Department of Water Resources, Sacramento, has been nominated to succeed Joe W. Kelly as president for a 1-year term beginning at the February, 1961, convention. He is now completing his second year as Institute vice-president.

Joining ACI in 1926, Mr. Tuthill has participated in nearly every phase of Institute activity. He has been a member of the Board of Direction and Technical Activities Committee, and has served as chairman of Committee 611, Inspection of Concrete, and Committee 614, Recommended Practice in Measuring, Mixing, and Placing Concrete. He is currently serving on five ACI technical committees: 116, Nomenclature; 207, Properties of Mass Concrete; 609, Consolidation of Concrete; 611, Inspection of Concrete; and 621, Aggregates.

Well known to JOURNAL readers, Mr. Tuthill has been a frequent contributor of technical papers for many years. He has prepared or collaborated on about 15 papers for the JOURNAL. His paper, "Concrete Operations in the Concrete Ship Program" (ACI JOURNAL, January 1945) earned him the ACI Construction Practice Award. In 1956 he and William A. Cordon were awarded the Wason Medal for the most meritous paper of the year for their joint contribution, "Properties and Uses of Initially Retarded Concrete."

Mr. Tuthill assumed his present position in 1956. Previously, he had been associated with the Bureau of Reclamation in Denver for 17 years, and was chief of the Concrete Laboratory when he resigned to join the California State Department of Water Resources.

Since graduation from Oregon State College in 1920, Mr. Tuthill has been

Lewis H. Tuthill

continuously engaged in design and construction work on irrigation, water supply projects, and the building of dams, with the technique and control of concrete as his primary interest.

For Vice-President

Roger H. Corbetta, president, Corbetta Construction Co., Inc., New York, has been a staunch supporter of ACI activities since joining in 1941. He is currently a member of the Board of Direction and ACI's representative on the Concrete Industry Board of New York City. Mr. Corbetta was a member of the ACI Building Committee, 1956-59. He was chairman of the local planning committee for the 1960 annual ACI convention in New York.

Mr. Corbetta has been identified with precast concrete structures and prestressing methods since 1920. He has pioneered in the field of thin shell concrete construction and is one of the nation's leading concrete contractors.

One of the founders of the Concrete Industry Board of New York City, he served as its chairman for 5 years, He is a trustee of Dutchess Community College in New York.

Through his firm he has directed a diversity of projects in many parts of this country as well as in Puerto Rico and Spain. He has visited and consulted on construction problems in Central and South America as well as in the middle eastern countries of Syria, Lebanon, Kuwait, Saudi Arabia, Iraq, and Iran.

Roger H. Corbetta

Raymond C. Reese, consulting engineer, Toledo, Ohio, was elected to a 2-year term as vice-president at the 1960 convention, and will continue in that office.

Board of Direction

Nominated to the ACI Board of Direction for 3-year terms beginning at the February, 1961, convention are: Arthur R. Anderson, G. E. Burnett, Chester P. Siess, and Anton Tedesko.

Arthur R. Anderson, partner, Anderson, Birkeland and Anderson, structural engineers, Tacoma, Wash., has been an ACI member for the past 10 years. He is currently a member of ACI Committee 324, Precast Reinforced Concrete, Thin Sections, and ACI-ASCE Committee 712, Precast Structural Concrete Design and Construction.

Dr. Anderson has specialized in static and dynamic strain measurements, stress analysis, and prestressed construction.

Receiving his BS from the University of Washington in 1934, an MS in 1935, and DSc in 1938 from the Massachusetts Institute of Technology, Dr. Anderson became a research assistant in civil engineering at MIT. In 1938-39 he served as structural designer for Aug. Klonne Steel Co., Dortmund, Germany, on the design of welded steel highway bridges for the Reichsautobahn. He then returned to MIT as a research associate working on the development of instrumentation and techniques for the study of the behavior of materials under high rates of loading.

From 1941-46 Dr. Anderson acted as head of the technical department of Cramp Shipbuilding Co., Philadelphia, engaged in the design and construction of naval vessels. The following 5 years he engaged in consulting engineering and research work in concrete and experimental stress analysis.

Dr. Anderson was co-founder of Concrete Technology Corp., Tacoma, in 1951. The organization specializes in research, technology, product development, and production of high strength

concrete structural members and precast and prestressed concrete.

Arthur R. Anderson

After 5 years as a partner in the consulting firm of Anderson and Anderson, specializing in prestressed concrete structural design, he became a partner in Anderson, Birkeland and Anderson, structural engineers.

G. E. Burnett, assistant chief research engineer, U.S. Bureau of Reclamation, Denver, has been an ACI member since 1946. He is currently a member of the Technical Activities Committee and chairman of the newly formed TAC Committee on Monographs. Formerly chairman of both, Mr. Burnett is now a member of Committee 612, Recommended Practice for Curing Concrete, and Committee 616, Coatings for Concrete.

G. E. Burnett

Mr. Burnett graduated from the University of Utah in 1930 and has been employed in the Engineering Laboratories Branch of the USBR since 1936. Much of his early work was devoted to investigation of curing compounds for concrete and coatings for concrete. He has been assistant chief of the Division of Engineering Laboratories since 1952 and was recently named assistant chief research engineer. Prior to joining the USBR he was employed in the research laboratory of Standard Oil Co. of New Jersey in a refinery at Aruba, N.W.I.

Mr. Burnett has contributed a number of technical papers for publication in the JOURNAL. Committee reports issued during his chairmanship include: Recommended Practice for the Application of Portland Cement Paint to Concrete, Guide for Painting Concrete, and Curing Concrete.

Chester P. Siess, professor of civil engineering at the University of Illinois, Urbana, has been active in ACI affairs since 1938.

After his graduation from Louisiana State University in 1936, Professor Siess worked as soils engineer in the testing laboratory of the Louisiana Highway Commission. In 1937 he became special research graduate assistant in theoretical and applied mechanics at the University of Illinois where he received his MS in 1939 and his PhD in 1948.

Chester P. Siess

From 1939-41 he worked with the Chicago Department of Subways and Superhighways and the New York Central Railroad, returning to the University of Illinois in 1941.

Much of Professor Siess' research has been published in technical journals. He has won many honors including the ACI Wason Medal for Most Meritorious Paper, 1949, which he coauthored with N. M. Newmark; the 1956 Concrete Reinforcing Steel Institute Award (with N. M. Newmark); and the 1956 ASCE Research Prize.

Long active in ACI committee work he currently serves as chairman of ACI Committee 115, Research, and ACI-ASCE Committee 321, Design of Reinforced Concrete Slabs. He is former chairman and now a member of ACI Committee 208, Bond Stress; former chairman of Committee 215, Fatigue of Concrete; and a member of ACI Committee 318, Standard Building Code.

Anton Tedesko for years has served the Roberts and Schaefer Co., engineers of New York and Chicago, in a consulting practice on major industrial, institutional and government projects; he is now a vice-president of the company, in New York.

His name is identified with many wide-span structures and early prestressed concrete work in America:

Anton Tedesko

arenas such as the Denver Coliseum, prestressed concrete bridges, airplane hangars for the armed services, the shells of the St. Louis Lambert Airport terminal, and more recently of a department store in Denver. Among the first to advocate the use of industrial production methods in shell roof construction, he was directly responsible for the design and development of construction methods for a great number of projects. Recently he has been engaged in major basic programs for the Ballistic Missile Division of the U.S. Air Force involving missile launching facilities.

Dr. Tedesko is a consultant to Headquarters U.S. Air Force and as such served in the evaluation and correction of important construction failures.

Dr. Tedesko is a member of the Reinforced Concrete Research Council and is chairman of its Task Committee on Composite Construction. He is chairman of ACI Committee 334, Concrete Shell Structures, and a member of Committee 314, Rigid Frames for Buildings and Bridges, and Committee 315, Detailing Reinforced Concrete Structures. He is a former director of the ASCE Illinois section.

Educated in Austria, Dr. Tedesko holds the degree of civil engineer and Doctor of Science in engineering from the Institute of Technology of Vienna, Austria.

Nominating Committee

Members of the Nominating Committee in addition to Chairman Ketchum were Phil M. Ferguson, University of Texas, Austin; Ernst Gruenwald, Lone Star Cement Corp., New York; W. C. Hansen, Universal Atlas Cement Division of U.S. Steel Corp., Gary, Ind.; Eivind Hognestad, Portland Cement Association, Skokie, Ill.; Douglas McHenry, Portland Cement Association, Skokie, Ill.; Walter H. Price, U.S. Bureau of Reclamation, Denver; and Kenneth B. Woods, Purdue University, West Lafayette, Ind.

Twenty candidates have been chosen for the 1960 Nominating Committee, five of whom will be elected. Candidates are:

Harold Allen	G. M. Nordby
D. L. Bloem	O. O. Phillips
W. A. Cordon	Milos Polivka
J. G. Dempsey	M. E. Prior
J. R. Janney	Emil Schmid
T. B. Kennedy	C. H. Scholer
C. E. Kesler	J. N. Thompson
Ralph Kluge	E. W. Thorson
R. C. Mielenz	I. L. Tyler
Eric Molke	C. A. Willson

55-YEAR INDEX

Reminder—Members and subscribers recently received a special introductory offer by mail to reserve copies of the forthcoming ACI 55-Year Index at a special reduced price. The offer is good only until **Nov. 15, 1960**, for members and subscribers residing in the United States and Canada, and (because of mail delays) until Mar. 15, 1961, for those residing elsewhere. **Payment**, by check, money order, or bank transfer **must accompany** the order, which is not complete unless the **order form** and addressed **mailing label** are also returned.

When the Index comes off the press in December it will be priced at $9.00 ($4.50 to ACI members). Non-ACI members may reserve a copy now at the regular price if they desire.

When temperatures drop, Columbia Calcium Chloride in the mix keeps concrete jobs moving.

TEMPERATURE: 48°F JOB: ON SCHEDULE

with Columbia Calcium Chloride in the mix

Faced with winter concreting and a time-tight schedule? Columbia Calcium Chloride in the mix any time the temperature drops below 70°F helps prevent lag in strength gain. Your jobs keep *moving* in spite of low temperatures. You get these specific time-and-money-saving benefits:

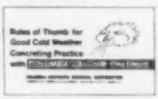
HIGHER EARLY STRENGTH—At 40°F, the strength of Columbia Calcium Chloride treated concrete is as great in 1 day as untreated concrete is in 2 days. At 3 days, the strength is equal to 7-day strength of untreated concrete.

FASTER INITIAL SET—Under normal conditions, Columbia Calcium Chloride reduces initial set from 3 hours to 1 hour. At lower temperatures, the accelerating effects are even more pronounced. You get finishers on and off the job faster and avoid costly overtime.

EARLY FORM REMOVAL—The use of Columbia Calcium Chloride means forms can be moved faster. Total time required for the job is cut.

Get all these Columbia Calcium Chloride cold weather benefits on your next job. Just add Columbia Calcium Chloride at the rate of 2% by weight of cement. Want technical help? Our calcium chloride engineers are at your service. For their assistance, just contact our District Sales Office nearest you or write to our Pittsburgh Office.

Write for free folder: "Rules of Thumb for Good Cold Weather Concreting Practice."



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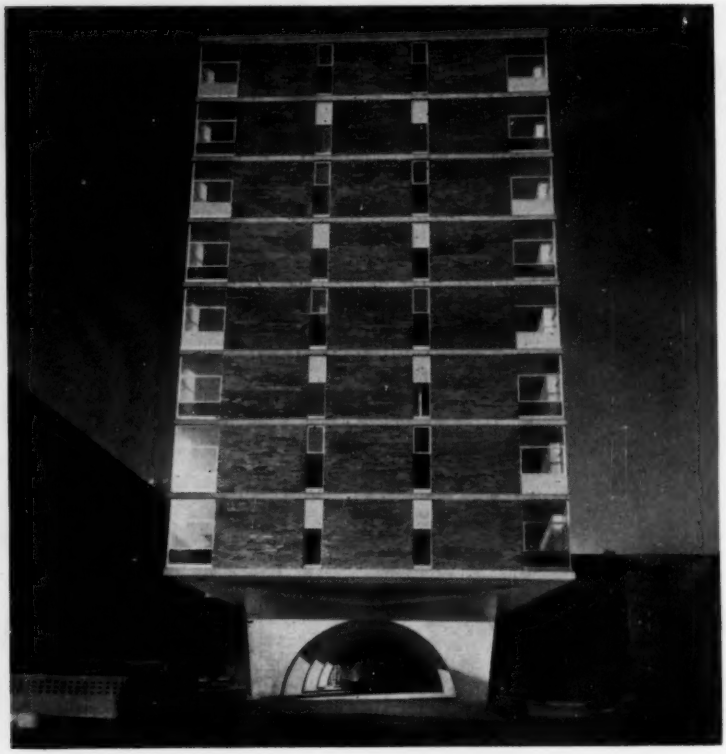


Reinforced CONCRETE Arches

Support Luxury Apartment Building

Proving that you can do the unusual with reinforced concrete is this luxury apartment building designed by Tasso Katselas. In this design, the entire first floor is supported directly on reinforced concrete arches which serve as an open colonnade leading to stairways and elevators. At ground level, the arches also support reinforced concrete columns extending the full height of the building, from which flat slab floors with continuous cantilevers extend around each side of the structure.

Only with reinforced concrete can you achieve such freedom of design. Before you build any type of building, investigate this more flexible medium of construction.

Neville House Apartments,
Pittsburgh, Pa.
Architect: Tasso Katselas, Pittsburgh, Pa.
Structural Engineers: Gensert, Williams & Associates, Cleveland, Ohio
Contractor: Gratziano Construction Company, Pittsburgh, Pa.
Photos by Jay-Bee Studios

Concrete Reinforcing Steel Institute
38 South Dearborn St., Chicago 3, Ill.

Positions and Projects

ACI technical committee appointments

Listed below are committee members who have recently accepted appointment to ACI technical committees. Included are *new* appointments only.

Committee 323 — Prestressed Reinforced Concrete — ACI-ASCE

Jack R. Janney
The Engineers Collaborative
Chicago, Ill.

Committee 331 — Structures of Concrete Masonry Units

C. E. Mason
Besser Co.
Alpena, Mich.

Mr. Mason was inadvertently listed as a newly appointed member of Committee 323 instead of Committee 331 on p. 5 of the October, News Letter.

ACI participates in trade missions program

ACI is one of many organizations participating in the trade missions program of the U.S. Department of Commerce. Technical societies and the business press contribute the 1000-unit library which accompanies each trade mission. These libraries, which include business publications, trade journals, and directories representative of virtually every industry in the United States, are used by the mission members to provide specific information needed during their interviews. After the mission, the library remains for continuing use by the U.S. embassy or consulate or a local group identifying United States sources of supply and in describing American products and processes having business potential there.

The program was selected by the Freedoms Foundation at Valley Forge to receive the George Washington Honor Medal "for its missionaries for capitalism program."

Southern California Chapter draws record attendance

Hot weather concrete construction was the subject under discussion at the September 15 dinner meeting of the ACI Southern California Chapter, bringing out a record audience of about 150.

The factors that create problems in hot weather construction with concrete,

Tentative program for 1961 ACI convention

The Technical Activities Committee has announced a tentative technical program for the 57th annual meeting of ACI to be held Feb. 20-23, 1961, at the Chase-Park Hotel, St. Louis, Mo.

The first 1½ days will be devoted to technical committee meetings with a Technical Committee Progress session on Tuesday afternoon, February 21. A number of brief committee reports will be presented at this session on work underway and progress within various committees.

Five technical sessions have been planned with several sessions running concurrently through Thursday, February 23. The first general session opens Wednesday morning. Design and Construction session and Materials session will run concurrently on Wednesday afternoon. Concurrent sessions on Thursday morning will feature Committee 216 Symposium on Fire Resistance and a Design session. Committee 115 will hold its annual research forum on Thursday afternoon.

Prominent American and European authorities will participate in the program presenting approximately 20 technical papers.

the means for control and methods recommended, were outlined by a three-man panel.

Everett A. Jumper, engineer of materials, Consolidated Rock Products Co., discussed mix proportion considerations, the selection of materials, the effects produced by various admixtures and additives, the desirable limits of temperature of the concrete and of elapsed time prior to placing and finishing, and the need for uniformity and consistency control.

William E. Bradford, vice-president, Robert J. Hiller Corp., outlines the problems that face the finishing organization in handling concrete, particularly in flat work, under hot weather conditions. He stressed the importance of organizing the finishing crew with sufficient trained personnel to accomplish a proper finishing job when the setting time is greatly accelerated, the necessity for completion of the finish and application of protection before shrinkage cracking may take place, and the use of fog spray to prevent too rapid evaporation from the surface. He also pointed out that the use of retarders, though often necessary, may extend the finishing time and involve extra finishing cost.

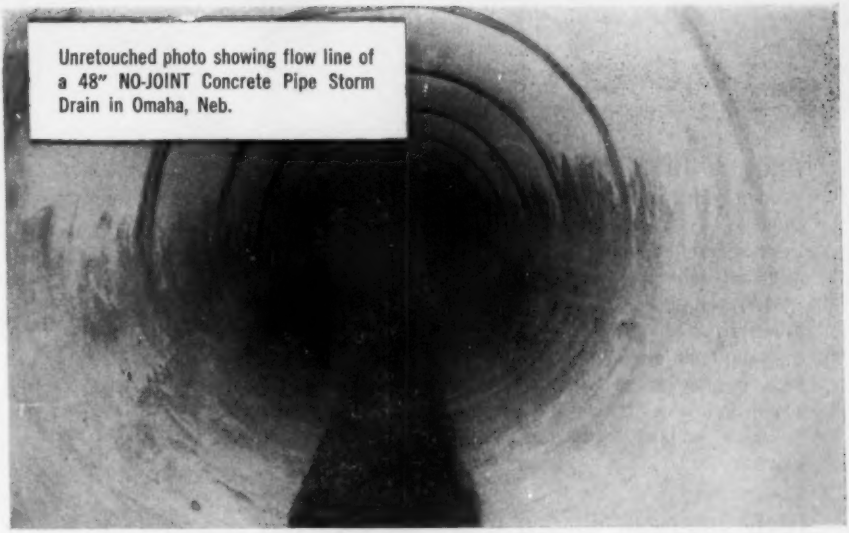
Stephen J. Collins, chief construction engineer, Albert C. Martin & Associates, discussed curing and protection, and pointed out that good curing is even more essential to the production of good concrete in hot weather than under any other condition. He cited continuous moist curing with water as the standard of comparison, and urged that for the best quality of work, 1 or 2 days of moist curing should precede the application of other curing treatments, such as sealed paper cover or sprayed membrane. He stated a preference for two thin applications of sealing membrane rather than one heavy application, feeling that the two-coat method produces a more uniform cure and minimizes pin holes. He urged greater use of fog spray at the critical stage in the finishing and protection of concrete laid in hot weather.

A general discussion followed the talks by the three speakers; among the points brought out was the fact that fog spray treatment has been used successfully on a number of jobs. Other interesting points were cited in comments from the floor. An experience was related concerning two jobs laid at the same time and with all conditions identical except that one job was equipped for and used fog spray early curing. It was stated that the work cured by the fog spray method had shown a complete absence of cracking after many months, while the other job was badly defaced by cracks.

It was stated that while fog spray nozzles have in general shown a tendency to corrode and to have a short useful life, it appears that equipment is or will be available in which that defect is being overcome. It seems evident that fogging will be more generally used when known to be fully practical.

A point of interest regarding the use of retarders in hot weather was brought to attention. It was stated that a most desirable condition for retarding the set is in the case of concrete which must be hauled for some distance from plant to point of use. A mixture properly treated with retarder under such a condition will be ready for finishing when placed and finishing will then involve no added expense; in fact, without a retarder finishing might be a serious problem with the time and temperature elements involved.

President Dewain R. Butler presided at the chapter meeting and Robert E. Tobin as program chairman presented the speakers and moderated the discussion.



Unretouched photo showing flow line of a 48" NO-JOINT Concrete Pipe Storm Drain in Omaha, Neb.

Smoother Flow through NO-JOINT Concrete Pipe!

Contractors: Are you making the *profit* you should on concrete pipe installations? Here is how you can *increase profit,* and still have *low bid* on the big jobs.

When you install *100% perfectly bedded* NO-JOINT Conduit you produce a *highest quality* product at *lowest cost.* You *eliminate* the high cost of *back filling and tamping* to the spring line. There's no hauling and handling of pipes, less trenching and backfilling. Black top and labor cost is *greatly reduced.* There are *no pipe inventories* to maintain; *no capital investment inventories* to tax!

Engineers: Cast-in-place, Roman arch design NO-JOINT Concrete-Pipe can *save up to 30%* on your concrete pipe dollar! Saves on maintenance, too, because there are *no joints to spring leaks!*

NO-JOINT Conduits have the *smoothest flow line;* efficient, clean-line design all the way! *Hundreds of miles* of NO-JOINT pipe now in use for storm drains, sanitary outfall sewers, culverts and irrigation projects. Sizes from 24" up to 72" ID.

Flexural strength of extruded NO-JOINT Concrete Pipe is *uniform* throughout its length. Bearing loads are *uniformly resisted* and *joint leaks eliminated.*

NO-JOINT CONCRETE PIPE CO

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P. O. BOX 609A
YUBA CITY, CALIFORNIA
U. S. A.

Territories: Many profitable NO-JOINT territories are still available. Write or phone for information on licensed territory or rental of NO-JOINT equipment today.

Free Catalog: Large, colorful catalog graphically describes the NO-JOINT Cast-in-place process. Write for your free copy today.

Another New York skyscraper has been reinforced with USS American Welded Wire Fabric. This is Rockefeller Center's new forty-eight story Time & Life Building, an outstanding example of contemporary architectural design. The exterior steel columns are encased in stone-faced concrete which project from the walls and serve to accent the vertical sweep of the tower.

The frame supports short span, lightweight concrete slabs reinforced with USS American Welded Wire Fabric. Each slab is 8'0" long and 4" thick. When asked why the fabric-reinforced short-span design was selected for this structure, W. B. Scofield, partner in the structural engineering firm of Edwards & Hjorth, said "This system provides first-class, fireproof construction with a long record of satisfactory service in addition to its proven economy, speed of construction, and occupancy flexibility."

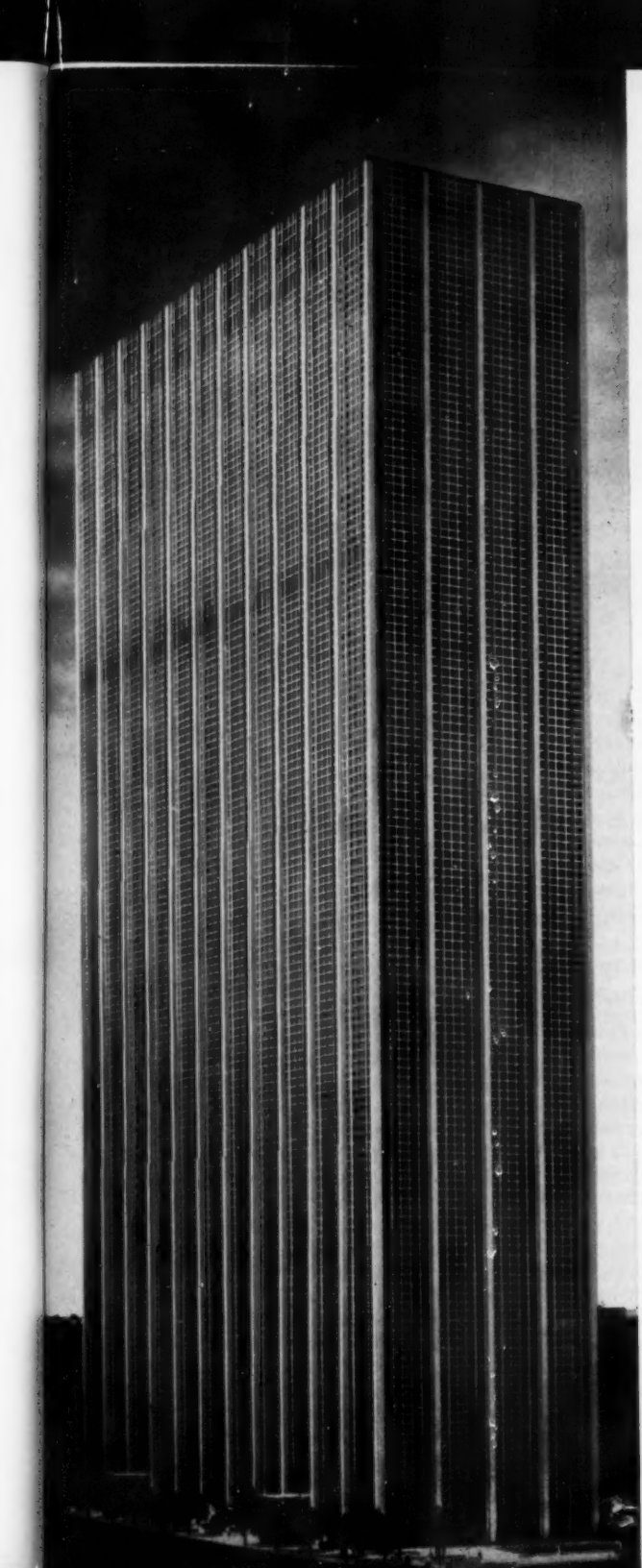
USS American Welded Wire Fabric was also used to reinforce the concrete fireproofing encasement of the columns, girders, and beams. Fabric is excellent for this application because the small, closely spaced members reinforce this thin concrete best. In addition, fabric is easily shaped to fit the contours and is sufficiently rigid to maintain the required shape.

Please write American Steel & Wire, Dept. 0379, 614 Superior Avenue, N.W., Cleveland, Ohio or contact our nearest sales office for complete information on these or any other uses of USS American Welded Wire Fabric.

USS and American are registered trademarks

Short-span fabric-reinforced floor system in Rockefeller Center's Time & Life Building. American Welded Wire Fabric was furnished in long rolls and merely unrolled perpendicular to the beams and on top of the forms. It was draped from the top of the slab over the beams to the bottom of the slab at mid-span. Thus, the reinforcement is in position to best resist both positive and negative moments. The economy of steel placement is apparent. In total, over six million square feet of short span slabs reinforced with Welded Wire Fabric have been used in New York's Rockefeller Center.

Owners: Rockefeller Center, Inc. and Time Inc.
Architects: Harrison & Abramovitz & Harris
Structural Engineers: Edwards & Hjorth
General Contractor: George A. Fuller Company, John Lowry, Inc.
Fabric Distributor: Fireproof Products, Inc.



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a product is made of
modern, dependable Steel.

American Steel & Wire
Division of
United States Steel

USS

Columbia-Geneva Steel Division,
San Francisco, Pacific Coast Distributors
Tennessee Coal & Iron Division,
Fairfield, Alabama, Southern Distributors
United States Steel Export Company,
Distributors Abroad

ACI headquarters attracts world-wide visitors

The Institute headquarters building has been included in the itinerary of countless engineers, architects, and students passing through Detroit the past few months.

The international interest in ACI is further evidenced with guests from seven foreign countries having signed the visitors' register recently. Architects, engineers, professors, and students from Australia, England, Finland, France, India, Nigeria, Switzerland, and Uruguay have been generous in their comments on the building. Among prominent overseas ACI members signing the register were H. A. Llewellyn, consulting engineer, Sydney, NSW, Australia, and A. M. Neville, lecturer in engineering at the University of Manchester, England, currently professor of civil engineering at the Nigerian College of Technology, Zaria, Nigeria.

Members and guests from across the United States ranging from California to New Jersey have inspected the headquarters building during the summer.

University of California to hold two prestressed conferences

University of California Extension and the Department of Engineering will present two conferences on prestressed concrete in November in Los Angeles and San Francisco. The 2-day meetings will be held November 14-15 at the Biltmore Hotel in Los Angeles, and November 21-22 at the Sheraton-Palace Hotel in San Francisco.

As a sequel to the 1957 World Conference on Prestressed Concrete these two western conferences will examine the architectural and engineering aspects of the use of prestressed concrete in buildings. Emphasis will be placed, in presentations and in panel discussions, on recent developments in industry and solutions to problems worked out by architects and engineers with extensive experience in prestressed concrete applications.

T. Y. Lin, professor of civil engineering, University of California, Berkeley, is conference chairman. The conference is being presented in cooperation with Prestressed Concrete Manufacturers Association of California, a chapter of the Prestressed Concrete Institute, and World Conference on Prestressed Concrete organization.

Dundee Cement staff appointment

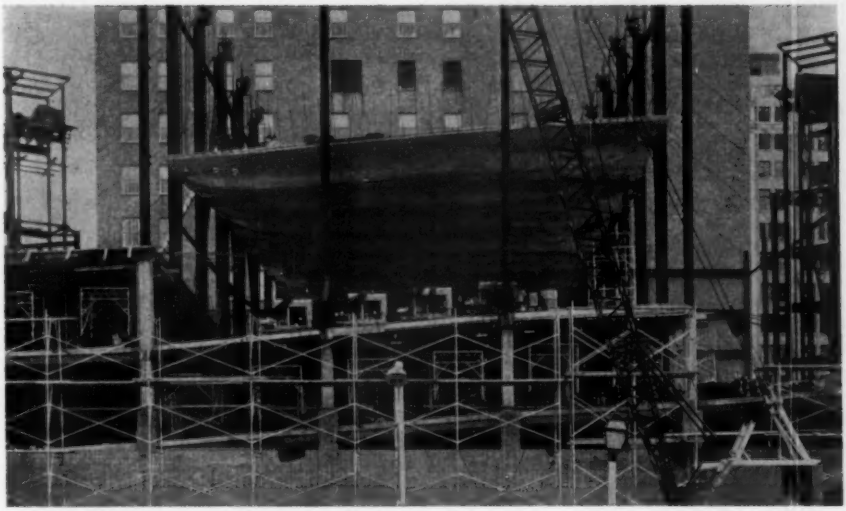
John D. MacDouglass has been appointed a sales representative for Dundee Cement Co., Dundee, Mich., in the Ohio district office at Cleveland and Edwin D. Sweeney has been appointed sales representative at Columbus, Ohio.

Prestressed concrete symposium held

The Engineers' Society of Milwaukee and the Prestressed Concrete Institute sponsored a symposium on prestressed concrete, September 23-24, in Milwaukee, Wisconsin.

The program, covering design, specification, and inspection of prestressed concrete, was presented by three veterans in the field of prestressing; Jack R. Janney and Richard C. Elstner, both members of The Engineers Collaborative, a Chicago organization of consulting structural engineers; and Karl Roesser, structural engineer for the Portland Cement Association, Chicago.

The above speakers are all active in ACI technical committee work. Mr. Janney is chairman of ACI-ASCE Committee 712, Precast Structural Concrete Design and Construction, and a member of ACI-ASCE Committee 323, Prestressed Reinforced Concrete. Mr. Elstner is a member of ACI-ASCE Committee 326, Shear and Diagonal Tension, ACI Committee 314, Rigid Frames for Buildings and Bridges, and Committee 339, Allowable Stresses in Reinforcement. Mr. Roesser is a member of ACI Committee 322, Design of Structural Plain Concrete.



Warped lift slabs for parking garage

Warped lift slabs are being raised to form the automobile ramps of Prudential Insurance Co.'s 500-car public garage now under construction in Newark, N. J. The 190 ft long slabs, 11 in all and weighing about 270 tons apiece, will form the roof and upper floors of the parking garage. A pair of electric consoles with a keyboard of gages and control valves ride atop the slabs and actuate hydraulic jacks that raise them into position. All 11 slabs are warped to some degree and the sensitivity of control, to a tenth of an inch, permits them to be lifted at the exact angle at which they were cast.

Voorhees Walker, Smith, Smith & Haines, New York City, are the architects; Frank Briscoe Co., Newark, the contractor; and Lift-Slab Corp. of New York, the subcontractor.

Zackrison named chairman of BRAB advisory board

Harry B. Zackrison, Sr., supervisory general engineer, Office of the Chief of Engineers, Department of the Army, Washington, D.C., has been appointed 1960-61 chairman of the Building Research Advisory Board, a unit of the Division of Engineering and Industrial Research, National Academy of Sciences — National Research Council.

In earlier years Mr. Zackrison was best known in the engineering profession for his work in the development of building codes for structural steel, timber, and reinforced concrete. In recent years he has distinguished himself in virtually all phases of engineering involved in construction. As chief of the Engineering Division for Military Construction, Office of the Chief of Engineers, since January, 1952, he has been responsible for the establishment of engineering criteria and for engineering performance on a $14 billion world-wide construction program carried out by the Corps of Engineers for the Department of the Air Force and the Army.

Mr. Zackrison is currently a member of three ACI committees: 216, Fireproofing or Fire Protection of Structures; 314, Rigid Frames for Buildings and Bridges; and 318, Standard Building Code.

ACI *Proceedings* on microfilm

Institute members and JOURNAL subscribers may obtain complete ACI JOURNAL, V. 30, July 1958 through June 1959 (*Proceedings* V. 55), including discussion and indexes, on microfilm for $6.15.

This is an economical means of maintaining a file of ACI material with the minimum storage space.

Proceedings V. 46 (September 1949 to June 1950) through *Proceedings* V. 54 (July 1957 through June 1958) are also available.

For additional price information and orders, write University Microfilms, 313 N. First St., Ann Arbor, Mich.

Horn appoints Mericle midwest branch manager

A. C. Horn Companies, a division of Sun Chemical Corp., New York, has appointed M. J. Mericle as midwest branch manager with headquarters in Chicago. Mr. Mericle joined the Horn firm in 1959.

Valore joins Sika Chemical as director of research

Rudolph C. Valore, Jr. has joined Sika Chemical Corp., Passaic, N.J., as director of research. He was previously director of research and development for Texas Industries, Inc., Dallas for 5 years and for 17 years prior was with the National Bureau of Standards in Washington, D.C., as materials engineer.

In addition to his committee work with numerous technical organizations, Mr. Valore is currently serving on five ACI committees. He is chairman of ACI Committee 623, Cellular Concrete, and a member of Committee 115, Research, Committee 116, Nomenclature, Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete, and Committee 716, High Pressure Steam Curing.

Thomas Concrete Pipe assumes new firm name

Thomas Concrete Pipe Co., Oklahoma City, announced a firm name change to the American-Marietta Co., Concrete Products Division, Oklahoma City.

Columbia Cement establishes Columbus sales office

Columbia Cement Corp., Pittsburgh, has announced the transfer of its general sales offices from Zanesville, Ohio, to new headquarters in Columbus, Ohio. The move establishes the Columbus headquarters in the proximity of the firm's cement marketing area and the Ohio State Highway Department. Columbia Cement is a wholly-owned subsidiary of Columbia-Southern Chemical Corp. and one of the largest suppliers of portland cement in Ohio and the Ohio River Valley.

Second World Conference on Earthquake Engineering

Papers on earthquake engineering from nearly 20 countries were presented at the Second World Conference on Earthquake Engineering held July 11-18, 1960 at Tokyo and Kyoto, Japan.

The main part of the conference was divided into five sessions and a conference summary. The sessions were devoted to soil and foundation conditions; analysis of structural response and instruments; seismicity and earthquake ground motions; earthquake resistant design, construction and regulations; and recent strong motion earthquakes and resulting damages. Papers presented covered research in the field of engineering seismology and earthquake engineering and new developments in aseismic design and construction practices in various seismic countries.

The conference was organized by the Science Council of Japan in cooperation with the Japan Society of Civil Engineers, the Architectural Institute of Japan, and the Seismological Society of Japan.

University of Colorado faculty promotions

James Chinn was promoted to professor and Donald G. Leitch was promoted to assistant professor in the department of civil engineering at the University of Colorado, Boulder, Colo.

Whitlock elected PCI president

Jacob O. Whitlock, president of Midwest Prestressed Concrete Co., Springfield, Ill., was elected president of the Prestressed Concrete Institute at its annual convention, September 27-30, New York.

Robert J. Lyman, Atlas Structural Concrete, Inc., El Paso, Tex., was elected vice-president; and Robert A. Matthews, Precast Industries, Inc., Kalamazoo, Mich., was named secretary-treasurer.

New directors elected were: Harold R. Hutchens, Carter-Waters Corp., Kansas City, Mo.; W. C. Givens, Capitol Concrete Corp., Jacksonville, Fla.; Robert H. Singer, Ben C. Gerwick, Inc., San Francisco; Harry Edwards, Leap Concrete, Inc., Lakeland, Fla.; Edward Schechter, Stressteel Corp., Wilkes-Barre, Pa.

Charles B. Kiesel, Jr., Raymond International, Inc., New York, and Elmer D. Clark, Superior Sand & Gravel, Phoenix, Ariz., were re-elected to the board, and Ezra C. Knowlton, Utah Sand & Gravel Products Corp., Salt Lake City, continues as a director.

The meeting featured world renowned engineers on the subject of prestressed concrete, discussing new techniques and methods in this over $300 million industry. Representatives attending the meeting from Belgium, Canada, Austria, England, Italy, and Russia participated in the program.

Two professors from the USSR Academy of Construction and Architecture spoke on Russian manufacturing methods and techniques of pretensioning and prestressing: B. G. Skramtaev discussed the electrothermic method of pretensioning bar reinforcement of precast reinforced concrete. This method is now in large scale operation in Russian concrete plants, and is said to be simpler and more economical than the mechanical method. V. V. Mikhailov described recent USSR developments in the mechanization and automation of producing prestressed concrete.

ACI member Arthur R. Anderson, partner of Anderson, Birkeland and Anderson, Tacoma, Wash., spoke on new possibilities in design for economical prestressed concrete highway brid-

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ges at a session devoted to "Creative Engineering in Prestressed Bridges."

Another ACI member, Lev Zetlin, consulting engineer, and visiting professor of civil engineering at Manhattan College, New York, described a new concept of pretensioning in prestressed concrete pavements at one of the technical sessions. Dr. Zetlin pointed out that a portable abutment makes it possible to construct a continuous pavement prestressed in two directions, and the length of the pavement between joints may be as long as 1200 ft.

Curing Concrete with Electricity

Electric-resistance heating as a method of curing concrete bridge decks in winter has been used successfully by Penzel Construction Co., Jackson, Mo., on a bridge in Perry County, Mo.

Carl Penzel, president, said use of the electric heating cables, 100 ft long and for 110-v operation, resulted in a saving in costs and an opportunity "to sleep on cold, windy nights." Each 500-watt cable used on the job produced heat of 17 Btu per ft.

The insultated cables are covered by wet burlap and a 2-in. thick Fiberglas blanket. Thermostats are set at 80 deg.

Penzel used a thermostatically controlled liquefied petroleum gas furnace for heat under the bridge deck.

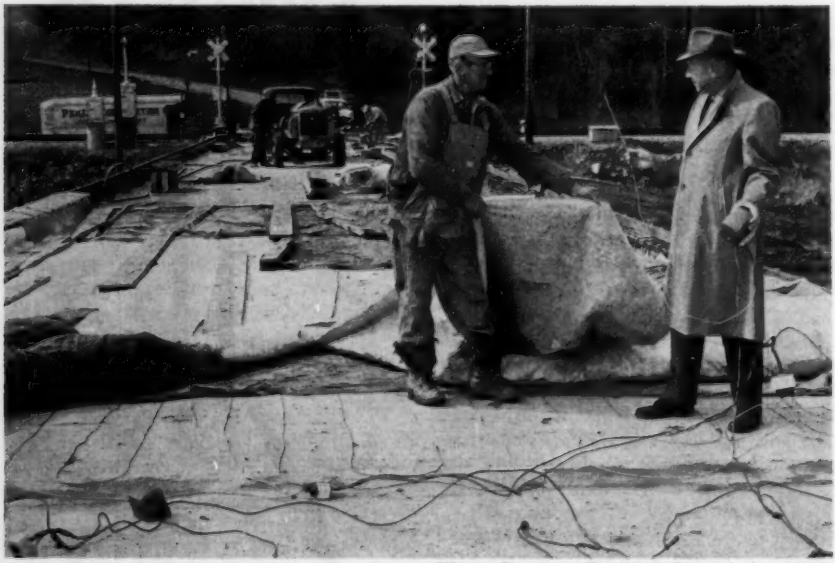

Photo Courtesy Portland Cement Association

Carl Penzel, Jackson, Mo., contractor (right), discusses with his foreman, Alvin Nagel, placing of electric heating cables for winter curing of concrete bridge decks

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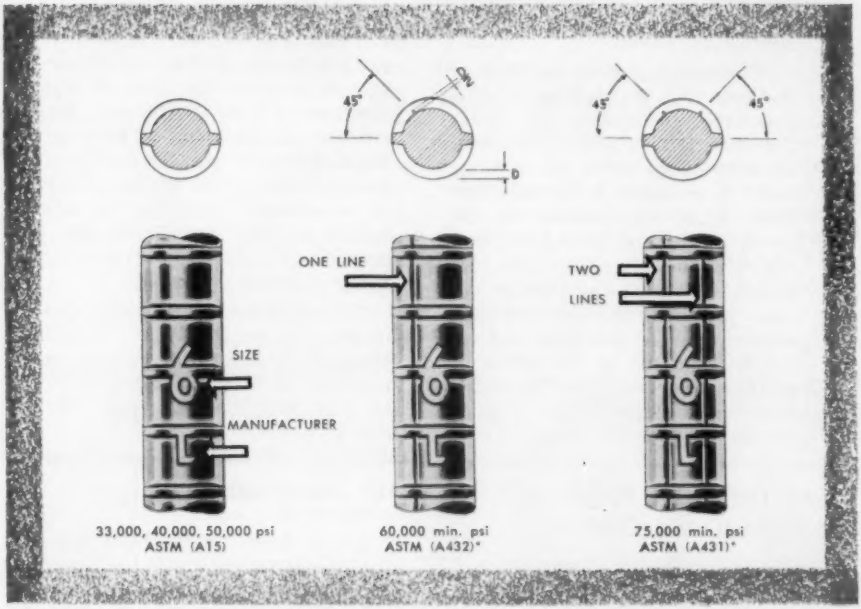

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Tax relief to British members

The British Commissioners of Inland Revenue have approved the American Concrete Institute for the purposes of Section 16, Finance Act, 1958. ACI members who qualify for tax relief under that section may deduct all of their annual dues from income subject to tax under Schedule E. The following is quoted from the letter of the Senior Principal Inspector of Taxes:

"Commencing with the year to 5 April 1961, a member who is assessable to income tax under Schedule E in respect of the emoluments of an office or employment is entitled to a deduction from those emoluments of the whole of the annual subscription which is due and payable by him to the society in the income tax year provided that — (a) the subscription is defrayed out of the emoluments of the office or employment, *and*, (b) the activities of the society so far as they are directed to all or any of the following objects — (i) the advancement or spreading of knowledge (whether generally or among persons belonging to the same or similar professions or occupying the same or similar positions); (ii) the maintenance or improvement of standards of conduct and competence among the members of any profession; (iii) the indemnification or protection of members of any profession against claims in respect of liabilities incurred by them in the exercise of their profession; are relevant to the office or employment, that is to say, the performance of the duties of the office or employment is directly affected by the knowledge concerned or involves the exercise of the profession concerned.

"A member of the society who is entitled to the relief should apply to his tax office as soon as possible for form P358 on which to make a claim for the relief due to him."

First Industrial Building Exposition and Congress

Sixty-one speakers, drawn from 15 states, and representing virtually every type of industry, will address sessions of the first Industrial Building Exposition and Congress at the New York Coliseum, December 12 through 15.

Two seminars which may be of interest to ACI members are scheduled for Tuesday afternoon, December 13: The Use of Prefabricated Components: Lift Slab, Curtain Walls; and Advances in the Use of Concrete: Precast, Prestressed, and Shells.

The show and conference are the first ever staged to meet the needs of companies building new plants or modernizing existing structures. About 100 exhibitors will show products and services for industrial buildings of all types.

ACI monographs

continued from p. 2

related to their mission. Final approval for publication rests with the Technical Activities Committee.

Monographs may be divided into subject series such as design, research, construction, or other categories although they need not be written as part of an established series. These publications will aim to develop a thorough understanding of the subject reported on. They will be tailored to be of maximum value to the practicing engineer, advanced student, contractor, and others having more than a superficial interest in and working knowledge of concrete. It is intended to publish the series as 6 x 9-in. books ranging from 75 to 150 pages. Diagrams, charts, drawings, photos, and tables will be included where required.

Prestressed concrete symposium conducted at Cooper Union

The Concrete Industry Board of New York in conjunction with Cooper Union and the Portland Cement Association sponsored a symposium on prestressed concrete in a series of three evening sessions October 11, 18, and 25.

Annual California street and highway conference

The 13th California Street and Highway Conference, presented annually by the Institute of Transportation and Traffic Engineering, and University Extension, University of California, will be held on the university's Berkeley campus, January 26-28. The conference program will cover new developments in matters ranging from policy and planning to safety, traffic engineering, construction, and maintenance. Chairman will be John Moran, chief engineer of Oakland, Calif.

Barney awarded contract for PIB graduate center

W. J. Barney Corp., New York, has been named general contractor for the Polytechnic Institute of Brooklyn's new graduate center, Long Island's first graduate school of engineering.

The facility with attendant research laboratories is being established at a cost of $1,600,000 on 25 acres of land. The approved plans call for two buildings with a total of 65,000 sq ft of floor space.

Bird views concrete structures in U.S.

Kenneth F. Bird, civil and structural engineer, Johannesburg, South Africa, recently spent 6 weeks in Washington, D.C., including visits to various reinforced and prestressed concrete structures. Mr. Bird is chairman of The Institution of Structural Engineers, Union of South Africa branch.

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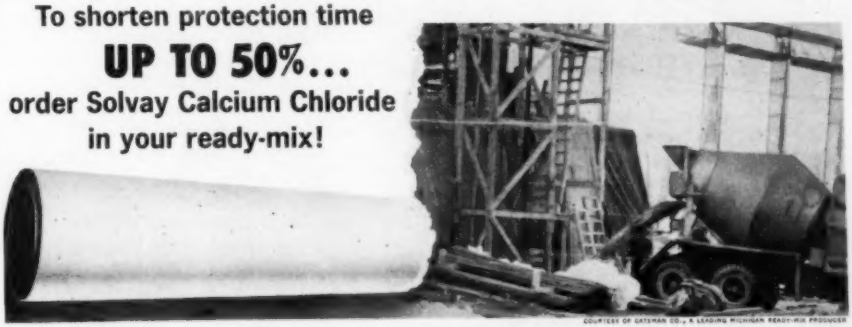

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*Speeds but does not change the basic action of portland cement. This use of calcium chloride is approved by Portland Cement Association, American Concrete Institute, Calcium Chloride Institute, leading highway departments.

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LOOKING AHEAD

Nov. 3-4, 1960—Annual Meeting, National Slag Association, Hotel Mayflower, Washington, D.C.

Nov. 14-15, 1960—Prestressed Concrete Conference, University of California Extension and the University of California Department of Engineering, Biltmore Hotel, Los Angeles, Calif.

Nov. 14-19, 1960 — Public Works and Municipal Services Congress and Exhibition, Olympia, London, England

Nov. 15-17—Fall Conferences, Building Research Institute, Shoreham Hotel, Washington, D.C.

Nov. 21-22, 1960—Prestressed Concrete Conference, University Extension and the University of California Department of Engineering, Sheraton-Palace Hotel, San Francisco, Calif.

Nov. 21-26, 1960—Symposium on Concrete Pavements, Buenos Aires, Argentina

Dec. 12-15, 1960—First Industrial Building Exposition and Congress, New York Coliseum, New York

Jan. 5-7, 1961 — FIP-RILEM Symposium on Injection Grout for Prestressed Concrete, Norges Tekniske Hogskole, Trondheim, Norway

Jan. 9-11, 1961—Annual Convention, Concrete Products Manufacturers Association of Quebec, Chateau Frontenac, Quebec City, Que., Canada

Jan. 9-13, 1961 — Annual Meeting, Highway Research Board, Washington, D.C.

Jan. 17-19, 1961—44th Annual Convention, National Crushed Stone Association, Hotel Americana, Bal Harbour, Fla.

Jan. 18-20, 1961 — Annual Convention, National Concrete Contractors Association, Diplomat Hotel, Hollywood-by-the-Sea, Fla.

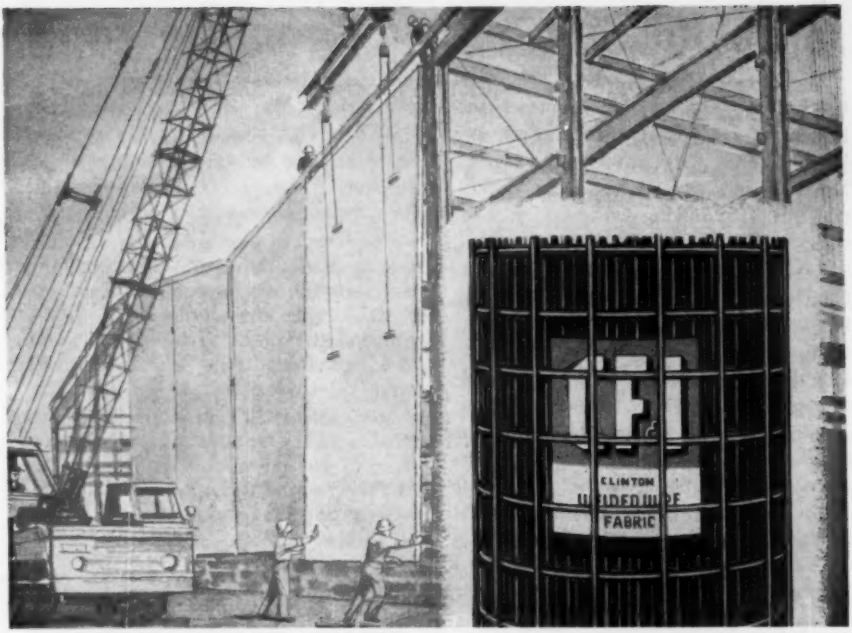
Jan. 23-26, 1961 — Annual Plant Maintenance and Engineering Show, International Amphitheatre and Palmer House Hotel, Chicago, Ill.

Jan. 23-26, 1961—31st Annual Convention, National Ready Mixed Concrete Association, Hotel Americana, Bal Harbour, Fla.

Jan. 26-28, 1961 — 13th California Street and Highway Conference, Instituute of Transportation and Traffic Engineering, and University Extension, University of California, Berkeley, Calif.

Jan. 30-Feb. 2, 1961—12th Biennial Concrete Industries Exposition, National Concrete Masonry Association, Cobo Hall, Detroit, Mich.

Feb. 20-23, 1961 — 57th Annual Convention, American Concrete Institute, Chase-Park Plaza Hotel, St. Louis, Missouri.



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Mechanization Cuts Concrete Time

Slattery Contracting Co., Inc., contractor for building an elevated section of the Cross-Bronx Expressway in New York City, saved time and labor through use of a specially built rig for distributing concrete for the deck.

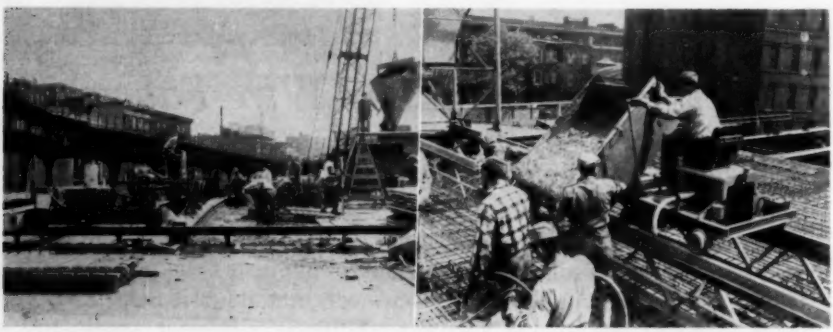
John Saunders, superintendent, and Charles Pretch, mechanic, fabricated, mostly from spare parts and scrap steel, a device to ride on rails and distribute stiff-mix concrete evenly across the 30-ft width of each roadway.

It consisted of a 2-cu yd hopper on one side, into which ready-mixed concrete was hoisted from below, and a motorized 1 cu yd dump truck that moved transversely on tracks and distributed concrete ahead of a mechanical finisher.

The device was originally designed to be pushed by the finisher, but was further developed to operate on its own gasoline power.

Operated by one man, the rig eliminated hand labor for spreading the concrete and made a conventional mechanical spreader unnecessary. It also reduced the time for placing each 138 ft long panel from 7½ hr, by usual methods, to 5 hr.

Mechanical finishing produced the close surface tolerances required on the job. Saunders said he expected to further reduce paving time on future jobs by applying the experience gained on this one.



Mechanical finishing of deck panel (left) follows distribution of ready-mixed concrete by a specially built rig (right) on the Slattery Contracting Co., Inc., part of the elevated Cross-Bronx Expressway in New York City. The dump truck is filled from a hopper charged by crane from below and then the dump truck moves transversely on tracks to distribute the concrete evenly across the deck. Carl Penzel, Jackson, Mo., contractor (right), discusses with his foreman, Alvin Nagel, placing of electric heating cables for winter curing of concrete bridge decks

Full coverage of the ACI 13th Regional Meeting, Oct. 31-Nov. 2, 1960, Tucson, Ariz., will be published in the January, 1961, JOURNAL.

Prescon plant opens in Hawaii

Prescon of the Pacific, Ltd., Honolulu, Hawaii, has been formed for the fabrication and sale of tendons as used in the Prescon system of post-tensioning prestressed concrete. Ben Hayashi is president of the new company.

The plant, which will serve the Hawaiian Islands and other Pacific areas has been equipped with Prescon machinery. Karl Middendorf, developer of the Prescon systems, has recently returned from a 30-day trip getting the operation underway.

Melleby moves to Norway

Ole Melleby, formerly of Denver, Colo., has moved to Oslo, Norway, where he will establish his own consulting firm.

Steel Scaffolding and Shoring Institute organized

The Steel Scaffolding and Shoring Institute, a national nonprofit organization with headquarters in Cleveland, has been formed by a group of leading manufacturers of these products.

The institute's objectives are to promote and establish recommended criteria and inspection procedures for, and collect and disseminate information concerning, steel scaffolding and shoring in the interest of the general public welfare and safety and the steel scaffolding and shoring industry.

Membership in the institute is open to organizations who manufacture steel scaffolding or steel shoring in the continental United States. Associate membership is available to independent distributors of these products, steel, and other suppliers to the industry.

New book tells

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How . . .

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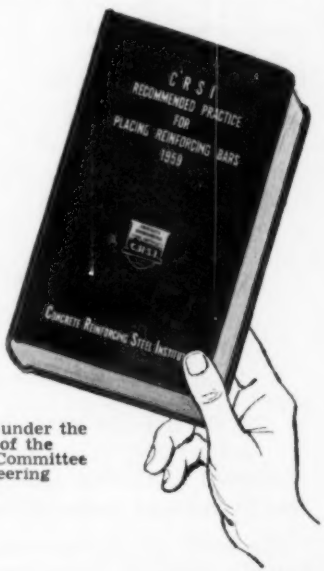

Prepared under the direction of the C.R.S.I. Committee on Engineering Practice.

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Errata

The following corrections should be made in "Determinations of Membrane Stresses in Elliptic Paraboloids Using Polynominals," by L. Fischer, which appeared in the October, 1960, JOURNAL.

p. 436 — in the formula for t in Eq. (3e), change "$\partial^2F/\partial x\partial y$" to "$-\partial^2F/\partial x\partial y$."

p. 437 — in the last line of Eq. (3g), change "Eq. (3f) (c)" to Eq. (3f) (b)."

p. 437 — in the first line of the last paragraph, change "Eq. (3c)" to "Eq. (3e)."

* * *

The following corrections should be made in "General Formulas for Membrane Stresses in Hyperbolic Paraboloidical Shells," by Felix Candela, which appeared in the October, 1960, JOURNAL.

p. 359 — in the second line of the second equation for v_x, change "$5/5\ k \sin \omega x_1$" to "$-5/4\ k \sin 2\omega_1$."

p. 360 — in the second line of Eq. (19c), change "$1/2\ Z_1 y$" to "$1/2\ Z_1 x$"

p. 361 — in the second line below Eq. (22c) change "$\theta = \omega/2$" to "$\phi \approx \omega/2$"

p. 364 — correct the following equations to read:

$$\bar{\sigma}_x = \sigma_\beta \frac{\cos(\beta - \alpha)}{\sin \beta} + (\tau_\beta - \tau) \frac{\sin(\beta - \alpha)}{\sin \beta} \quad (25a)$$

$$\bar{\sigma}_y = -\sigma_\beta \frac{\cos \beta}{\sin(\beta - \alpha)} - (\tau_\beta + \tau) \frac{\sin \beta}{\sin(\beta - \alpha)} \quad (25b)$$

p. 366 — in Table 1, Column Headings 1, 2, and 3, change Y, X, and Z, to y, x, and z respectively.

National Concrete Contractors fourth annual convention

The fourth annual convention of the National Concrete Contractors Association will be held January 18-20 at the Diplomat Hotel, Hollywood-by-the-Sea, Fla.

Clinton H. Chalmers

Clinton H. Chalmers, a partner in the firm of Chalmers and Borton, Hutchinson, Kan., died recently at the age of 72.

He and John Borton formed the partnership in 1926 and in the past 34 years the firm has built more than 1600 grain elevators in 26 states ranging from small country stations to huge terminal facilities. Most of the work centered in the midwest grain states but recently ranged to New York, California, and Florida.

Mr. Chalmers was born at North Bend, Neb. He was graduated in 1910 from the University of Nebraska School of Engineering. Active in numerous organizations, Mr. Chalmers had been an ACI member for the past 10 years. At the time of his death he was a member of ACI Committee 714, Concrete Bins and Silos.

Manley W. Sahlberg

Manley W. Sahlberg, perhaps best known for his leadership in efforts to make California schools safe from earthquakes, died recently in San Francisco.

Mr. Sahlberg, 68, was principal structural engineer in charge of the San Francisco office of the schoolhouse section, California State Division of Architecture, and was recognized as one of the leading structural engineers in California. He had worked for the state for 15 years and had headed the San Francisco office since 1952. Earlier he had been in private practice in Sacramento, where he designed the Elks building and the civic auditorium.

Mr. Sahlberg graduated from the University of California, Berkeley, in 1917. He served as a lieutenant in the Army Corps of Engineers in France during World War I. Active in a number of technical organizations he had been an Institute members since 1954.

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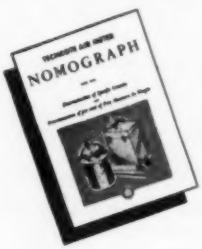

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Serafin joins Dewey and Almy

Frank G. Serafin has joined the Dewey and Almy Chemical Division, W. R. Grace & Co., Cambridge, Mass., as a junior research chemist in the construction chemicals and rock products chemicals laboratory.

Costinett joins BRI technical staff

John H. Costinett, Jr. has joined the Building Research Institute staff, Washington, D.C., in the capacity of associate staff engineer. He will work with Harold Horowitz, assistant director for technical programs, on the establishment of a new BRI service devoted to abstracting and indexing unpublished academic building research, in addition to other duties.

Mr. Costinett was formerly a materials engineer with General Services Administration and prior to that was chief of the District of Columbia engineering laboratory of the Corps of Engineers.

Precast Concrete Floor, Roof Units

MINIMUM STANDARD REQUIREMENTS FOR PRECAST CONCRETE FLOOR AND ROOF UNITS (ACI 711-58). This ACI Standard presents requirements for single units or multiple element assemblies, to be used in conjunction with ACI 318-56. Covers materials, design principles, manufacture including curing and handling, testing of completed units, installation plans, and special provisions for holes and openings in members. Design chapter discusses dimensions, allowable deflection, structural concrete topping, reinforcement anchorage and location, and use of lightweight concrete. 12 pp. Price $1.00, 50¢ to ACI members.

AMERICAN CONCRETE INSTITUTE PUBLICATIONS

Adams elected director of Huron Cement

Charles M. Adams, vice-president in charge of operations of Huron Portland Cement Co., Detroit, was elected to the board of directors at the annual meeting held recently.

At a stockholders' meeting prior to the annual meeting all incumbent directors were re-elected to the board of directors and authorization to increase the number of directors to six was given. The board of directors reelected all incumbent officers. They are: Paul H. Townsend, chairman; H. Ripley Schemm, president; Charles L. Laude, vice-president in charge of sales; and Earl W. Denby, secretary and treasurer.

Crichton opens consulting office

Robert M. Crichton has opened his consulting engineering office in Hood River, Ore. He will specialize in water and sewer systems, drainage and flood control, roads and streets, site planning, industrial buildings, and foundations. He recently resigned as city engineer in Hood River.

Calaveras awards construction contracts

Calaveras Cement Co., a division of The Flintkote Co., has awarded the M & K Corp., San Francisco, the contract for the construction of the manufacturing portion of a new plant north of Redding, Calif. The $4,355,000 bid covers installation of crushers, raw and clinker storage, mill buildings, kiln, and auxiliary equipment.

Homan & Lawrence Engineering Co., also of San Francisco, was awarded the contract for construction of raw storage silos, cement storage silos, and packhouse after submitting a low bid of $1,585,000.

The new Calaveras plant, which will have a capacity of 1,500,000 bbl of cement annually, is scheduled to begin operation in September, 1961.

Membership in the American Concrete Institute

To facilitate prospective members in joining the Institute, membership application forms are provided. Present Members may aid by bringing these forms to the attention of those who may profit from membership advantages. All who have an interest in concrete are eligible for membership. The grades of membership are described below.

Members have at hand in Institute publications the most complete fund of knowledge on concrete. The ACI Journal provides them with the latest information and ACI special publications provide them with the complete picture of specific problems. Through conventions, and regional and area meetings, they are afforded the opportunity of meeting those whose experiences provide the new information, and of exchanging ideas with them.

ACI's world-wide membership is growing in extent and participation —traveling a common road toward better, more economical and durable concrete structures. ACI provides a common ground in the search for and use of new "working tools" in concrete design, manufacture, and erection—and its interpretation.

Individual Members	(U.S. and Possessions, Canada, Mexico, Central America, and West Indies)	**$20.00**
Individual Members	(All other foreign countries)	**16.00**
Corporation Members		**65.00**
Junior Members—nonvoting (under 28)		**10.00**
Contributing Members		**135.00**
Student Members—nonvoting (under 28)		**5.00**

Please enclose remittance with application (cut here)

Board of Direction, American Concrete Institute **Date**_____
P. O. Box 4754, Redford Station
Detroit 19, Michigan

The undersigned hereby applies for admission to the American Concrete Institute as ☐ *Individual* ☐ *Corporation* ☐ *Contributing* ☐ *Junior* ☐ *Student Member and agrees to be governed by the Charter and Bylaws.*

Name and complete mail address of proposed membership (Address to which Journal is to be mailed—please letter)_____

_____

For Corporation Membership, ACI representative will be _____

Date of Birth (Juniors and Students only) _____
Year Month Day

College or University attending (Students only) _____

Signature_____

Month & Year of Graduation (Students Only)

(Proposed by) please print

For our records, please complete both sides of application.

NOTES on Membership Classification

The ACI advertising and editorial departments need definite information concerning job title, classification, business affiliation, and principal responsibility of all members. Thus, the information requested on the classification form below is especially important.

The applicant should designate his title or position such as: construction superintendent, research director, technician, draftsman, structural engineer, inspector, plant superintendent, highway engineer. The occupation of the applicant should be that which is **most frequently** performed. The principal responsibility of the applicant is that specific area of his job for which he is **primarily concerned.**

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ACI editors want to know your interests—they welcome suggestions as to what you'd like to see discussed in the ACI Journal and of the possible sources. Please attach a separate note or letter.

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Honor Roll

Jan. 1 — Sept. 30, 1960

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1 point for Student; 2 points for Junior; 3 points for Individual; 4 points for Corporation; and 5 points for Contributing.

Is your "ACImanship" showing? The continuing goal of the American Concrete Institute is to uncover new ideas and develop new techniques and improved standards covering all phases of concrete.

You can further these aims of the Institute by keeping the membership spiraling. Why don't you sign up one new ACI member this month?

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If you receive the JOURNAL regularly, we have your correct mailing address. **But is your job title and company affiliation as listed in the 1959 Directory still the same?**

Complete the coupon below (please print) and mail today to ACI Membership Division, P. O. Box 4754, Redford Station, Detroit 19, Michigan. **If we don't hear from you, we will assume your 1959 Directory listing is still current.**

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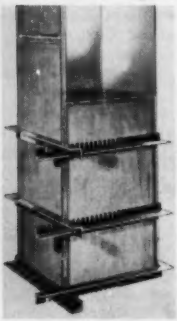
Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Square column forms



Deslauriers square column form system is made up of only three basic pieces: the column panels faced with heavy galvanized steel, the Econ - O - Clamps that square the forms, and the keys that quickly lock forms in assembled position.

The forms are light in weight and easy to handle; and the steel surfaces are said to strip fast and produce smooth concrete finish. Econ-O-Clamps are available separately for use with conventional plywood column forms. —Deslauriers Column Mould Co., Inc., 5036 W. Lake St., Chicago 44, Ill.

Nuclear gage for measuring granular materials

Nuclear gages have entered a new field — measurement of solid materials. The first dry solids mass flow density gage recently installed in a cement plant opens the application of radioactive sources as a control for materials in granular form moving through a production process. This includes cement, plastics, and minerals — materials whose specific gravity can be determined.

The key to the gage's high degree of accuracy and stability is said to be a unique conversion cell. This detector takes radiation emitted from radioactive sources such as cesium 137 and cobalt 60 and converts it directly to electrical current. This is amplified to operate recorders and meters for either automatic or operator control of the manufacturing process.

While gamma sources are used in most density and level gages, a beta radiation source, strontium 90, is used in the first dry solids flow gage.

The operation of this gage is based on the Bremstrahlung effect, where a beta ray hitting solid material causes x-rays of various energies to be emitted. These pass through the material to the conversion cell on the opposite side. The amount of x radiation reaching the cell is determined by the amount of material passing a given point at any instant.

The new gage can be installed with either a point or strip source. In the former, there is a single point where measurement is done. In the latter, source strips can be mounted one on top of the other for measurement over a distance.

Ohmart says beta sources will be employed where the velocity of the process is high and quantity small. Gamma sources will be used where velocity is slow and quantity high. Fast velocity is defined as movement at 10 ft per sec and slow is 3 ft per sec.

In the first installation, fine cement powder will be measured on a mass flow basis after it passes over an Air Slide and is falling through a gravity feed pipe into a kiln for final burning. The measurement will indicate how much material is reaching is kiln. The object is to get just the right amount of powder into the kiln. This will improve cement quality and production efficiency. —Ohmart Corp., Cincinnati, Ohio

Post-tensioning system

VSL post-tensioning system, used for many years in Europe, is now available in the United States through Colcrete Structures.

The following advantages of the system are cited by Colcrete: no fabrication of button heads; no exact precalculated cable length

necessary; unnecessary to grip wires in tensioning jack; cable capacities from 60 to 740 kips; due to positive wedge action no slippage occurs; multistage stressing and restressing made easy; larger diameter wire (0.315) is used, reducing the effect of wobble.

It is further claimed that units can be fabricated on job site thus simplifying shipment, storage, and handling problems. — Colcrete Structures, Inc., 10 East 40th St., New York 16, N.Y.

Aluminum pipe hoisting hose

Plastering and masonry contractors who pump mortar up several stories may be interested in a new 2-in. aluminum pipe for vertical "lift" from the machine to a higher story in the building.

The pipe is said to withstand bursting pressures up to 2000 psi, working pressures to 500 psi; to be rustproof, lightweight, and easy to transport and setup. Furnished in 10 ft section, threaded at both ends. — Goldblatt Tool Co., 1910 Walnut St., Kansas City 41, Mo.

Stud fastener

The Columbia-Matic "8" stud fastener, a stud driving revolver, sets eight powder driven fasteners from a single load and can drive 16 studs per min into concrete, steel, or wood according to Richmond Screw Anchor, Inc., newly appointed distributor for the unit. —Richmond Screw Anchor Co., Inc., 816 Liberty Ave., Brooklyn 8, N.Y.

Portable concrete plant

A unique low silhouette portable concrete batching plant provided fast, low cost erec-

tion on job site construction at Vandenburg Air Force Base, Santa Maria, Calif.

Aggregates are conveyor loaded to a 100-ton, three-compartment bin; 1500 cu ft of cement are stored in a separate silo; a batch conveyor transfers materials to mixer trucks. The plant will produce 32,000 cu yd of concrete. The plant was used in continuous slip-form concreting requiring up to 9 days of 'round-the-clock batching. The low silhouette is especially suitable to areas where height restrictions exist. Plant can be winterized. — Nobel Co., 1860 7th St., Oakland, Calif.

Automatic bending machine

The Swedish manufactured Alimak automatic bending machine, Type ST 52-36 is designed for rational bending of reinforcing bars. It is claimed that "Cam-iron" in stand-

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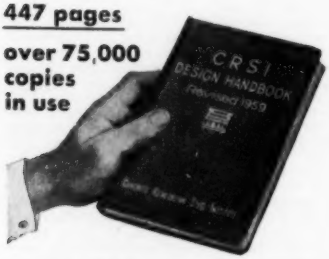

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ard qualities with diameters of 35 mm (1 2/5 in). or less are bent easily.

The unit can be adjusted to three different speeds 4, 8, and 12 rpm. By using different rollers, pins, and adjustable holding-on tools bends with radiuses of 30 mm (1 1/5 in.) or more can be made. Flat-iron and square-iron may also be bent. Bending may take place from left or right.

The desired bending angle is set on a graduated scale on the body. The bending disk is push button started. After bending, it goes automatically back to the starting point but may also be adjusted to manual starting and stopping. The machine is driven by a 7.5-hp electrical motor. The machine is 1100 (3 ft 8 in.) in height; weight, 400 kg (890 lb). —Alimak Verken, 14230 Skelleftea, Stockholm, Sweden

Concrete cylinder capping set

A basic cylinder capping set for preparing compression test specimens of concrete has been designed by Soiltest for laboratories

which need to cap relatively few concrete cylinders.

The outfit, for vertical type capping, includes vertical cylinder capper and capping plate, capping compound warmer (2 quart), capping ladle, 50 lb of concrete capping compound, and a cylinder carrier for 6 x 12-in. cylinders.

The capping sets are in conformity with ASTM C 31 and C 192 and AASHO T-23 and T-126. —Soiltest, Inc., 4711 W. North Ave., Chicago 39, Ill.

Reusable batter washer

The reusable Richmond batter washer is a new addition to the line of form ties, anchors, inserts, and accessories developed by Richmond Screw Anchor Co.

The batter washer has been designed to, whenever practicable, hold a Tylag at an angle up to 45 deg without need for expensive wedging. This is made possible by a slot in the haunch at the top of the batter washer which allows the Tylag to swing freely to the desired angle. Nail holes are provided for easy, accurate attaching to wales or strongbacks and multiple lumber grips on the underside prevent slippage when batter washer is not nailed. They are suited for Ty-down forms, battered walls, pier nose forms, and abutment corners. They are available for use with ½, ¾, and 1-in. Tylags. —Richmond Screw Anchor Co., Inc., 816-838 Liberty Ave., Brooklyn 8, N.Y.

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WILLIAM A. MAPLES
(Signature of editor)

Sworn to and subscribed before me this 3rd day of October, 1960.

JESSIE F. ALTENBERNT
(SEAL)
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